# INDEPENDENT FORENSIC TEAM REPORT
# OROVILLE DAM SPILLWAY INCIDENT

**JANUARY 5, 2018**

# SUMMARY

*The Oroville Dam spillway incident was caused by a long-term systemic failure of the California Department of Water Resources (DWR), regulatory, and general industry practices to recognize and address inherent spillway design and construction weaknesses, poor bedrock quality, and deteriorated service spillway chute conditions. The incident cannot reasonably be "blamed" mainly on any one individual, group, or organization.*

*During service spillway operation on February 7, 2017, water injection through both cracks and joints in the chute slab resulted in uplift forces beneath the slab that exceeded the uplift capacity and structural strength of the slab, at a location along the steep section of the chute. The uplifted slab section exposed the underlying poor quality foundation rock at that location to unexpected severe erosion, resulting in removal of additional slab sections and more erosion.*

*Responding to the damage to the service spillway chute necessitated difficult risk tradeoffs while the lake continued to rise. The resulting decisions, made without a full understanding of relative uncertainties and consequences, allowed the reservoir level to rise above the emergency spillway weir for the first time in the project's history, leading to severe and rapid erosion downstream of the weir and, ultimately, the evacuation order.*

There was no single root cause of the Oroville Dam spillway incident, nor was there a simple chain of events that led to the failure of the service spillway chute slab, the subsequent overtopping of the emergency spillway crest structure, and the necessity of the evacuation order. Rather, the incident was caused by a complex interaction of relatively common physical, human, organizational, and industry factors, starting with the design of the project and continuing until the incident. The physical factors can be placed into two general categories:

- Inherent vulnerabilities in the spillway designs and as-constructed conditions, and subsequent chute slab deterioration
- Poor spillway foundation conditions in some locations

A simplified overview of how human, organizational, and industry factors interacted with these two general categories of physical factors is given in Figure S-1, and is broadly outlined below.

The inherent vulnerability of the service spillway design and as-constructed conditions reflect lack of proper modification of the design to fit the site conditions. Almost immediately after construction, the concrete chute slab cracked above and along underdrain pipes, and high underdrain flows were observed. The slab cracking and underdrain flows, although originally thought of as unusual, were quickly deemed to be "normal," and as simply requiring on-going repairs. However, repeated repairs were ineffective and possibly detrimental.

The seriousness of the weak as-constructed conditions and lack of repair durability was not recognized during numerous inspections and review processes over the almost 50-year history of the project. Over time, chute flows and temperature variations led to progressive deterioration of the concrete and corrosion of steel reinforcing bars and anchors, with likely loss of slab strength and anchor capacity. There was likely also some shallow underslab erosion and some loss of underdrain system effectiveness, which contributed to increased slab uplift forces. The particularly poor foundation conditions at the initial service spillway chute failure location contributed to likely low anchor capacity and shallow underslab erosion.

Due to the unrecognized inherent vulnerability of the design and as-constructed conditions and the chute slab deterioration, the spillway chute slab failure, although inevitable, was unexpected.

Once the initial section of the chute slab was uplifted, the underlying poor quality foundation materials were directly exposed to high velocity flows and were quickly eroded. Undermining and uplift of other portions of the chute slab resulted in further removal of slab sections and more foundation erosion.

Although the poor foundation conditions at both spillways were well documented in geology reports, these conditions were not properly addressed in the original design and construction, and all subsequent reviews mischaracterized the foundation as good quality rock. As a result, the significant erosion of the service spillway foundation was also not anticipated.

Following the unexpected chute slab failure and erosion, and subsequent closure of the service spillway gates to examine the damage, delicate and difficult risk tradeoffs, involving myriad considerations, were necessary over the next few days in order to manage the incident. Either the gates would need to be re-opened, with the potential for further service spillway damage and/or damage to a transmission tower, or the lake levels would rise and the emergency spillway weir would be overtopped, with the potential for erosion at the emergency spillway. In addition, erosion had transported a tremendous amount of debris into the river channel, and the resulting high tailwater was threatening to flood the powerhouse. The decision-makers attempted to find a “sweet spot,” such that the service spillway would continue to be used, but with discharges no greater than necessary to just prevent the lake from rising above the emergency spillway weir.

There were decision points during the incident when discharge through the service spillway was specifically limited, even though risks to the powerhouse from further discharge were clearly diminishing. These decisions ultimately resulted in the lake rising high enough to initiate flow over the emergency spillway weir. The decisions were made with the best of intentions, but against the advice of civil engineering and geological personnel, who had by then recognized the poor bedrock conditions and the potential for unsatisfactory performance of the previously untested emergency spillway. In limiting service spillway discharge to reduce the likelihood of powerhouse flooding, the additional dam safety risk associated with use of the emergency spillway was not appropriately considered. Once the emergency spillway was allowed to overtop, this additional risk was soon realized, and the evacuation order became a necessary precaution.

There were many opportunities to intervene and prevent the incident, but the overall system of interconnected factors operated in a way that these opportunities were missed. Numerous human, organizational, and industry factors led to the physical factors not being recognized and properly addressed, and to the decision-making during the incident. The following are some of the key factors which are specific to DWR:

- The dam safety culture and program within DWR, although maturing rapidly and on the right path, was still relatively immature at the time of the incident and has been too reliant on regulators and the regulatory process.

- Like many other large dam owners, DWR has been somewhat overconfident and complacent regarding the integrity of its civil infrastructure and has tended to emphasize shorter-term operational considerations. Combined with cost pressures, this resulted in strained internal relationships and inadequate priority for dam safety.
- DWR has been a somewhat insular organization, which inhibited accessing industry knowledge and developing needed technical expertise.
- DWR's ability to build the appropriate size, composition, and expertise of its technical staff involved in dam engineering and safety has been limited by bureaucratic constraints.

In addition to lessons which are specific to DWR, as described in this report, the following are some of the general lessons to be learned by the broader dam safety community:

- In order to ensure the safe management of water retention and conveyance structures, dam owners must develop and maintain mature dam safety management programs which are based on a strong "top-down" dam safety culture. There should be one executive specifically charged with overall responsibility for dam safety, and this executive should be fully aware of dam safety concerns and prioritizations through direct and regular reporting from a designated dam safety professional, to ensure that "the balance is right" in terms of the organization's priorities.
- More frequent physical inspections are not always sufficient to identify risks and manage safety.
- Periodic comprehensive reviews of original design and construction and subsequent performance are imperative. These reviews should be based on complete records and need to be more in-depth than periodic general reviews, such as the current FERC-mandated five-year reviews.
- Appurtenant structures associated with dams, such as spillways, outlet works, power plants, etc., must be given attention by qualified individuals. This attention should be commensurate with the risks that the facilities pose to the public, the environment, and dam owners, including risks associated with events which may not result in uncontrolled release of reservoirs, but are still highly consequential.
- Shortcomings of the current Potential Failure Mode Analysis (PFMA) processes in dealing with complex systems must be recognized and addressed. A critical review of these processes in dam safety practice is warranted, comparing their strengths and weaknesses with risk assessment processes used in other industries worldwide and by other federal agencies. Evolution of "best practice" must continue by supplementing current practice with new approaches, as appropriate.
- Compliance with regulatory requirements is not sufficient to manage risk and meet dam owners' legal and ethical responsibilities.

Some of these general lessons are self-evident, and have been noted by others previous to the IFT's investigation of this incident. The question is whether dam owners, regulators, and other dam safety professionals will recognize that many of these lessons are actually *still to be* learned. Although the practice of dam safety has certainly improved since the 1970s, the fact that this incident happened to the owner of the tallest dam in the United States, under regulation of a federal agency, with repeated evaluation by reputable outside consultants, in a state with a leading dam safety regulatory program, is a wake-up call for everyone involved in dam safety. Challenging current assumptions on what constitutes "best practice" in our industry is overdue.

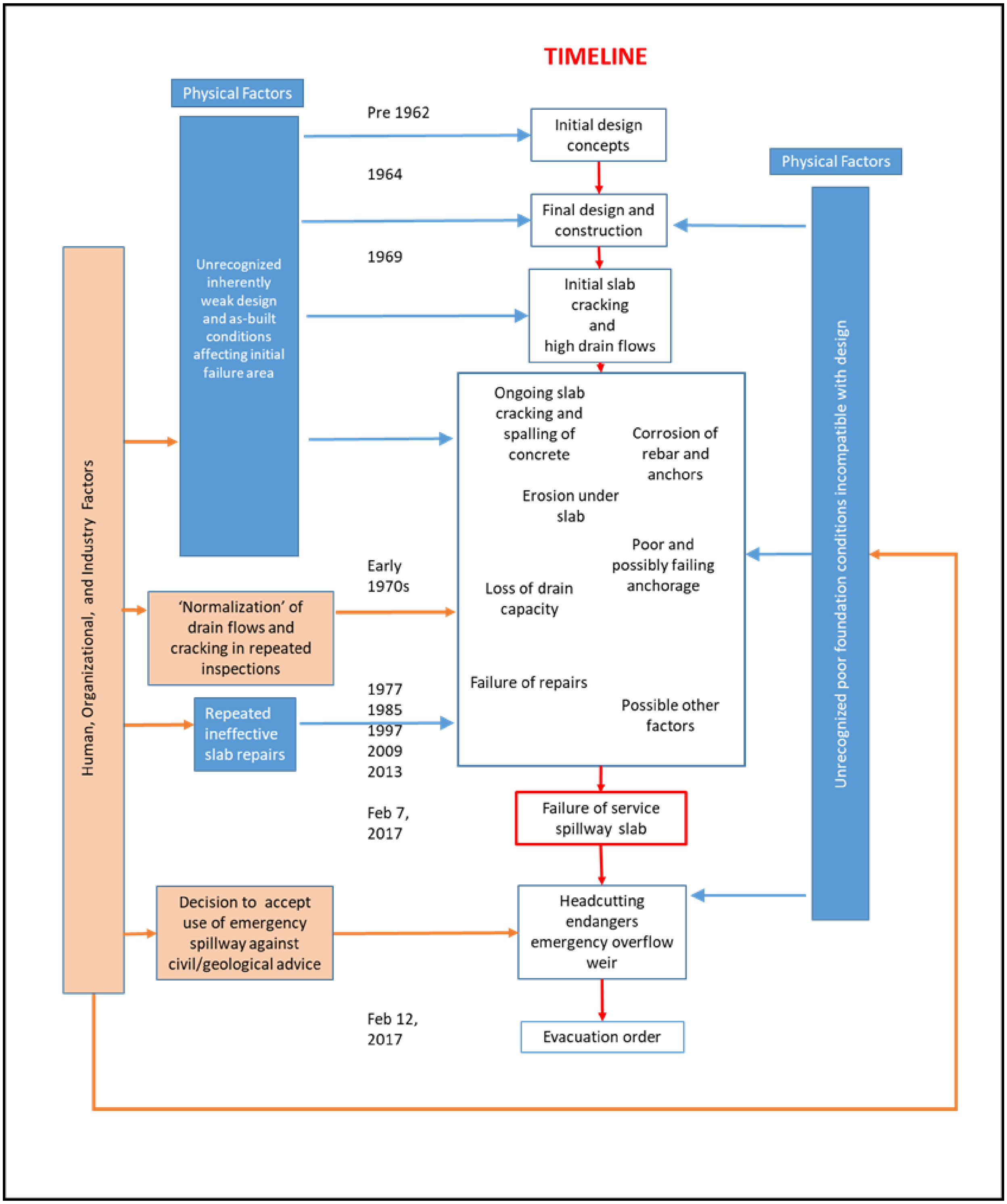


**Figure S-1: Overview of interacting factors leading to the Oroville Dam spillway incident**

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| | |
|---|---|
| ACI | American Concrete Institute |
| A-F | acre-feet |
| ASDSO | Association of State Dam Safety Officials |
| BOC | Board of Consultants |
| Caltrans | California Department of Transportation |
| CDSE | Chief Dam Safety Engineer |
| CEG | Certified Engineering Geologist |
| CFD | Computational Fluid Dynamics |
| cfs | cubic feet per second |
| D.GE | Diplomate, Geotechnical Engineering |
| D.WRE | Diplomate, Water Resources Engineering |
| DOE | DWR Division of Engineering |
| DSB | DWR Dam Safety Branch |
| DSOD | California Division of Safety of Dams (a DWR division) |
| DSPMP | Dam Safety Performance Monitoring Program |
| DSRB | Director's Safety Review Board |
| DWR | California Department of Water Resources |
| FCO | Flood Control Outlet |
| FERC | Federal Energy Regulatory Commission |
| FMEA | Failure Modes and Effects Analysis |
| ft | feet |
| gpm or GPM | gallons per minute |
| GPR | Ground Penetrating Radar |
| IC | Incident Command |
| ICOLD | International Commission on Large Dams |
| IFT | Independent Forensic Team |
| in | inch |
| ksi | thousand pounds per square inch |
| M | magnitude (earthquake) |
| No. | Number |
| O&M | DWR Division of Operations and Maintenance |
| OFD | DWR Oroville Field Division |
| PE | Professional Engineer |
| PFMA | Potential Failure Mode Analysis |
| PG | Professional Geologist |
| PhD | Doctor of Philosophy |
| PMF | Probable Maximum Flood |
| psi | pounds per square inch |
| PVC | Polyvinylchloride |
| Reclamation or USBR | U. S. Department of the Interior, Bureau of Reclamation |
| ROV | Remotely Operated Vehicle |
| RTS | Reservoir Triggered Seismicity |
| RVOS | River Valve Outlet System |

| | |
|---|---|
| SCADA | Supervisory Control and Data Acquisition |
| sec | second |
| SEED | Safety Evaluation of Existing Dams |
| Sta. | Station |
| STID | Supporting Technical Information Document |
| SWC | California State Water Contractors |
| SWP | California State Water Project |
| USACE | U. S. Army Corps of Engineers |
| USSD | United States Society on Dams |
| VCP | vitrified clay pipe |
| YCWA | Yuba County Water Agency |

# 1.0 INTRODUCTION

## 1.1 Team Formation and Authorization

After the Oroville Dam spillway incident in February 2017, the Federal Energy Regulatory Commission (FERC) required the California Department of Water Resources (DWR) to engage an Independent Forensic Team (IFT) to develop findings and opinions on the causes of the incident.

DWR asked two leading dam engineering and dam safety associations in the United States, the Association of State Dam Safety Officials (ASDSO) and the United States Society on Dams (USSD), to recommend members for the IFT. ASDSO and USSD recommended a six-member team, all of which were accepted by DWR, and subsequently engaged by DWR through separate contracts. The team members are:

John W. France, PE, D.GE, D.WRE – Team Leader and Geotechnical Engineer

Irfan A. Alvi, PE – Hydraulic Structures Engineer and Human Factors Specialist

Peter A. Dickson, PhD, PG – Engineering Geologist

Henry T. Falvey, Dr.-Ing, Hon.D.WRE – Hydraulic Engineer

Stephen J. Rigbey – Director, Dam Safety at BC Hydro, and Geological Engineer

John Trojanowski, PE – Hydraulic Structures Engineer

Resumes for all team members are presented in Appendix M of this report.

Contracts for all of the team members except Dr. Dickson were issued on April 8, 2017, and the IFT's work commenced at that time. Dr. Dickson's contract was issued a short time later and he joined the team's efforts.

Although the IFT members' contracts are with DWR, the IFT has carried out its work independently, and its efforts have not been directed or controlled by DWR. The IFT has also completed its work independently of all other organizations and agencies, including regulatory bodies, government agencies, consulting firms, water user groups, and professional societies, including the ASDSO and USSD. The IFT was not involved with the design and construction of spillway repairs after the incident, and the IFT worked independently of the Board of Consultants (BOC) which was formed to provide review and advice for the repair work.

DWR has supported the IFT by providing data from its files, assisting with arranging meetings and interviews with DWR personnel, and completing some investigative work requested by the IFT.

This report was not reviewed by DWR, the California Division of Safety of Dams (DSOD), or FERC at any point prior to its finalization.

## 1.2 Purpose of the Investigation

Before beginning its work, the IFT developed the following statement of purpose for the investigation:

> *To complete a thorough review of available information to develop findings and opinions on the chain of conditions, actions, and inactions that caused the damage to the service spillway and emergency spillway, and why opportunities for intervention in the chain of conditions, actions, or inactions may not have been realized. Evaluations of actions, inactions, and decisions for the various stages of the project (pre-design, design, construction, operations, and maintenance) will consider the states of practice applicable to the various time periods involved.*

## 1.3 Report Organization

This report is organized into eight sections and thirteen appendices.

The main report provides sufficient information regarding the incident to follow the physics of what happened, and why it happened in terms of both physical and human factors[1]. The main report also provides lessons to be learned.

The findings and opinions in the main report are supported by more detailed discussions and information in the appendices.

[1] In this report, the term "human factors" is intended to extend beyond individual factors and also include organizational and industry factors.

# 2.0 SCOPE AND METHODOLOGY OF THE INVESTIGATION

## 2.1 Focus and Limitations of the Investigation

The IFT's efforts were focused on the Oroville Dam service spillway chute and the emergency spillway, both of which suffered damage in the February 2017 incident. The IFT did not delve into issues related to the embankment dam, the service spillway headgate structure, or any other components of the Oroville facility. As the IFT reviewed the FERC-mandated Part 12D inspection reports and other documents, it noted that there have been some issues of concern raised related to the embankment dam and the service spillway headgate structure, but those issues are beyond the scope of the IFT's mission.

The IFT considered the emergency management of the incident only to the degree that it affected the activation of the emergency spillway. Detailed review of the emergency management process was again beyond the scope of the IFT's mission.

The IFT based the opinions and findings presented in this report in large part on information that was made available to the IFT during the investigation. As noted in Section 2.2 and Appendix J, the IFT attempted to cast a broad net to obtain pertinent information, but the amount of information available for Oroville Dam is very large, and the IFT cannot be certain that all pertinent information has been compiled. During the course of interviews and other communications, the IFT became aware of some items of pertinent information that had not been originally provided and requested copies of that information. During the last weeks of the investigation, some items of pertinent information were still being discovered. It must therefore be acknowledged that some items of pertinent information that could have impacted the IFT's opinions and findings could come to light after this report is issued.

The IFT had originally intended to submit a draft of the report to DWR for review and verification of factual information before finalization. However, to maintain independence from DWR, and because of concerns that information in a draft report would be made public prematurely, a draft was not provided to DWR before finalization. The IFT has endeavored to verify its interpretation of factual information, as well as information obtained in interviews and surveys.

## 2.2 Investigation Methodology

The IFT endeavored to complete as thorough an investigation and evaluation as practical to develop evidence for its findings and opinions. The IFT's work included:

- Identification of key documents related to contributors to the incident, and a thorough and critical review of relevant documents. Documents considered included:
    - Pre-design, design, and construction documents related to original design and construction of the project, including geologic reports, drawings, specifications, construction reports, and photographs.
    - Records of inspections and evaluations of the project by various entities including DWR Division of Operations and Maintenance (O&M), DWR Division of Engineering

(DOE), DSOD, FERC, and Director's Safety Review Boards (DSRBs)/FERC Part 12D Boards.

  - Potential Failure Mode Analyses (PFMAs) and DSRB/FERC Part 12D Board reports completed for the project.
  - Records of maintenance and surveillance monitoring activities, repairs, and modifications to the spillways.
  - Records of evaluations of issues related to the spillways.
  - Records of spillway operations, including photographic records.
  - Written reports, photographs, videos and other documentation of the February 2017 incident.
  - Results of investigations and evaluations completed by DWR and its consultants after the February 2017 incident.
  - Governance, guidance, and procedural documents for applicable dam safety, operation, and maintenance organizations associated with the project.
  - Notes taken during the incident in the Incident Command Center.

  A very large number of documents were initially provided to the IFT by DWR. As those documents were reviewed, the IFT identified and requested other documents which were subsequently provided by DWR. During the review process, the IFT found that some of the records related to the project are incomplete; this was particularly true for the records from the time of the original design and construction. The lack of particular records is noted at various places in this report and its appendices.

- Visits to the site to observe post-incident conditions.
  - Five of the six team members made an initial visit to the site on Thursday, April 13, 2017.
  - The sixth team member, Dr. Dickson, made separate visits to the site, including visits during forensic investigations of the service spillway chute, completed before chute sections were demolished for repairs.
- Meetings and interviews with individuals involved in various aspects of the spillways and the incident, or otherwise in a position to provide information relevant to the investigation. Individuals interviewed included current and retired DWR employees, DSOD employees, FERC employees, U. S. Army Corps of Engineers (USACE) personnel, and individuals associated with the original Oroville Dam design and construction. In total, more than 75 individuals were interviewed, either in person or by phone, and some individuals were interviewed more than once. Most interviews were in-depth and lasted more than an hour, in some cases much longer than an hour.

- Reviews of design details for 110 spillway chutes designed in the timeframe from about 1955 to about 1975, approximately ten years before to ten years after the Oroville Dam spillway design.
- Hydraulic analyses of flow characteristics for the Oroville Dam service spillway chute, possible leakage through slab cracks and joints, and the underdrain system flow capacity.
- Identification of investigations of the sections of the spillway chute remaining after the chute failure and of tree root characteristics; the investigations were subsequently completed by DWR and its consultants.
- A public request for information related to the incident, with an independent email box established to contact the IFT, which was publicized through the media, ASDSO, and USSD. Numerous emails were received, and several individuals who contacted the IFT were interviewed.
- A request for information which could be useful to the IFT, which was sent to all DWR employees. Several respondents sent emails to the same independent email box as described above, and several of them were interviewed.
- Two surveys of DWR personnel. One survey focused on educational and training background, professional experience and registrations, areas of expertise, and involvement in dam and spillway engineering and safety. The other survey was more general and solicited opinions regarding DWR organizational culture, internal communications and coordination, utilization of consultants, dam safety priority, budgets, schedules, and contributing factors to the February 2017 incident. There were about 100 respondents to each survey.
- IFT working sessions to collectively evaluate factual information and develop opinions; both face-to-face and web-based meetings were held.
- Preparation of a memorandum titled "Preliminary Findings Concerning Candidate Physical Factors Potentially Contributing to Damage of the Service and Emergency Spillways at Oroville Dam" [1] issued on May 5, 2017 to provide input to the spillway repair efforts.
- Preparation of an Interim Status Memorandum [2] issued on September 5, 2017 to publically share the IFT's findings as of that date.

Through an iterative process, the IFT applied a mix of fundamental inductive and deductive logic to the assembled information, and provides the resultant evidence and arguments in this report and the appendices. All of this work, which involved thousands of hours of collective effort by the IFT, has culminated in the preparation of this final report of the IFT's findings and opinions.

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# 3.0 BACKGROUND

This Section 3 provides background information on the Oroville Dam project, the spillways of Oroville Dam, the original design and construction of the spillways, the operational history of the spillways, the history of repairs to the service spillway chute, the history of drain flows at the service spillway, and the chronology of the February 2017 incident.

## 3.1 Oroville Dam Project Description

The following project description was developed principally from information in the Supporting Technical Information Document (STID) [3], including the 2014 PFMA [4].

Oroville Dam is a component of the Oroville-Thermalito Complex, which includes Hyatt Power Plant, Thermalito Diversion Dam and Power Plant, Fish Barrier Dam, Thermalito Power Canal, Thermalito Forebay, Thermalito Afterbay, and Thermalito Power Plant. The facility is a multi-use water resource project, constructed for the purpose of water conservation, power generation, flood control, recreation, and fish and wildlife management. It is located in the foothills of the Sierra Nevada Mountains above the Sacramento Valley, approximately five miles northeast of the City of Oroville, California. The Oroville-Thermalito Complex stores approximately 3.6 million acre-feet of water and has a generating capacity of 841 MWs combined from the three power plants.

The Oroville-Thermalito Complex itself is a component of the California State Water Project (SWP), which is the largest state-owned water storage and delivery system in the United States. The SWP provides water to about 25 million people and 750,000 acres of irrigated farmland in California. Planning of the SWP began mainly in the 1950s, with design and construction of most of the SWP completed in the 1960s and early 1970s. The Oroville Dam project was constructed in the 1960s, with the project completed in 1968.

The SWP is owned and operated by the California Department of Water Resources (DWR), which is part of the California Natural Resources Agency and was created in 1956. The SWP includes 22 dams. All of these dams are regulated by a separate division within DWR, the Division of Safety of Dams (DSOD). Eleven of the dams, associated with power production, are also regulated by the Federal Energy Regulatory Commission (FERC).

Oroville Dam is located at the upstream end of the Oroville-Thermalito Complex. Power can be generated by releasing water from Lake Oroville through the Hyatt Power Plant, or water can be released into the diversion pool impounded by the Thermalito Diversion Dam and the Thermalito Power Canal Headworks.

The Oroville Dam facility (see Figure 3-1) consists of an embankment dam, the Oroville Flood Control Outlet (FCO or service spillway)[2], the Oroville emergency spillway, the Hyatt Power Plant, the River Valve Outlet System (RVOS), and the Palermo Tunnel and Outlet. Oroville Dam

[2] In this report, the Oroville Dam Flood Control Outlet or FCO is referred to as the Oroville Dam service spillway, or simply as the service spillway.

is the tallest dam in the United States at 770 feet. The design embankment crest is at Elevation 922[3], and the maximum normal operating pool level is Elevation 900. For reference, the service spillway gate sill is at Elevation 813.6, and the crest of the emergency spillway overflow structure is at Elevation 901.


**Figure 3-1: Overview of Oroville Dam facility prior to the February 2017 incident**

The Oroville Dam service spillway, in particular the service spillway chute, and the Oroville Dam emergency spillway are the structures of interest in this investigation. Both spillways are described in Section 3.2 below.

[3] All elevations are reported in feet according the datum used on drawings and in DWR records.

## 3.2 Oroville Dam Spillway Descriptions

The following descriptions of the Oroville Dam spillways were developed principally from the as-built drawings [5] for the project.

### 3.2.1 Service Spillway

The service spillway is located on the right abutment of the main dam. It consists of an unlined approach channel, a gated headworks structure with an effective crest length of 140.7 feet, and a concrete-lined chute extending to just above the river channel (see Figure 3-2). The headworks structure has a total of eight top-seal[4] radial gates. It is comprised of two concrete monoliths, each containing four radial gates and five piers. The gates are identical, except for the trunnion support beams at the ends of the monoliths (Bays 1, 4, 5, and 8). The trunnion beams at the ends of the monoliths were modified in 2002 to eliminate single-sided trunnion girder cantilever action. The individual gates are 17 feet, 8 inches wide and 33 feet, 6 inches high. The upstream sill elevation of the service spillway headworks is at Elevation 813.6 and the tops of the gates (in the closed position) are at approximately Elevation 847. The chute downstream of the headgate structure is about 179 feet wide and just over 3,000 feet long, extending from the Elevation 811.80, at the downstream end of the headworks, to about 100 feet above the Feather River, a drop of about 500 feet. The first 1,000 feet of the service spillway chute downstream of the headworks slopes at about 5-2/3 percent, after which the chute transitions through a vertical curve to a much steeper slope of about 24.5 percent for the last 1,455 feet. Four large, reinforced concrete chute blocks or dentates are located at the downstream end of the chute, to disperse the spillway flow as it enters the river. Spreading the flow over a larger area than the concentrated flow from the chute decreases the amount of erosion in the river.

According to the 1970 *Report on Reservoir Regulation for Flood Control* [6], the service spillway discharge is 296,000 cfs, with the reservoir at Elevation 917 and all gates fully open.

### 3.2.2 Emergency Spillway

The emergency spillway is also located on the right abutment, to the right of the service spillway. The emergency spillway consists of two sections: a 930-foot long, gravity ogee weir on the left side, and an 800-foot long broad crested weir on the right side. The ogee weir section of the spillway is shown in Figure 3-3. The crests of both sections are at Elevation 901, which is 1 foot above the maximum normal operating reservoir level, Elevation 900. The maximum height of the emergency spillway crest structure is about 50 feet in the ogee weir section. Water flowing over the emergency spillway crest structure then passes over natural terrain to the Feather River. The emergency spillway was activated for the first time in the project's history during the February 2017 flood event and spillway incident.

[4] A top-seal radial gate is configured such that the top of the gate is below the maximum reservoir water surface and the upstream face of the gate is entirely submerged during times of high reservoir levels.

**Figure 3-2: Oroville Dam service spillway**


**Figure 3-3: Oroville Dam emergency spillway ogee crest structure**

According to the 1970 *Report on Reservoir Regulation for Flood Control* [6], the emergency spillway discharge is 350,000 cfs with the reservoir at Elevation 917, which corresponds to a depth of flow over the crest weirs of about 16 feet. For comparison, as discussed further below, during the February 2017 incident, the depth of flow over the crest weirs was about 1.6 feet. This corresponds to a discharge of about 12,500 cfs, which is only about 3 percent of the discharge cited in the 1970 report.

## 3.3 Service Spillway Chute Design and Construction

Understanding the service spillway chute design and construction is critical to understanding the chute failure in February 2017. This report section provides a summary of design and construction information. More detailed information is presented in Appendix A.

The IFT notes that available design and construction information was incomplete. Some detailed design calculations were not available for IFT review, including spillway chute slab design calculations. Construction reports were also not available for all time periods. While it is known that one or more Boards of Consultants (BOCs) were involved in design and construction, it is not known if all BOC reports from the design phase were available, and no BOC reports were available from the construction phase.

### 3.3.1 Chute Slab Thickness

The nominal service spillway chute slab design thickness was a minimum of 15 inches, although the chute slab thickness as constructed varied significantly. As discussed in Section 3.3.2 the chute slab thickness over drains was much thinner. In post-failure forensic studies [7], chute slab thicknesses at numerous locations other than at drains were found to vary from 14.5 to 81.6 inches, for portions of the slab which remained in place after the chute failure.

### 3.3.2 Chute Slab Drains

The design included an underdrain system consisting of herringbone drains beneath the slab, connected to collector pipes outside of and parallel to the service spillway chute walls. The herringbone drains were so named because each drain was oriented across the spillway with the pipes on either side of the spillway sloping downstream for drainage, from a point beneath the spillway centerline, as shown in Figure 3-4. The herringbone drains were nominally spaced at an interval of 25 feet along the flatter upstream section of the chute and at 20 feet along the steeper downstream section. When viewed in plan, the pattern of these drains looked like fish bones or herring bones, as shown in Figure 3-4, hence the name. Sets of herringbone drains connected to separate collector drain pipes on either side of the chute. The collector drain pipes extended downstream to outfall locations near the tops of the chute walls. To accommodate all of the herringbone drains, there were 12 collector drain pipes on each side of the chute, for a total of 24 collector drain pipes. The collector drain pipe outfalls are designated in this report as 1L to 12L and 1R to 12R, from upstream to downstream on the left and right sides, respectively.

In the original design/bid drawings, the herringbone drains were indicated as 4-inch diameter perforated vitrified clay pipes (VCPs) surrounded by gravel and the collector pipes were indicated as 8-inch and 6-inch diameter VCPs. However, construction records indicate that the herringbone drain size was increased to 6-inch diameter and the collector pipe size was increased to 12-inch diameter, which is consistent with the as-built drawings and the pipe sizes observed in the post-incident forensic investigations and during demolition. In both the design and as-built drawings the herringbone drains are not located entirely beneath the slab, but rather they protrude up into the slab section, as shown in Figure 3-6, taken from the as-built drawings. This location of the drain pipes substantially reduced the thickness of the slab immediately above the drains. As shown in Figure 3-5, the slab thickness above the drains was noted as a minimum of 7 inches. From available information, it appears that, in most locations, even when the as-constructed slab thickness increased, the drain pipe locations were maintained near the top of the slab.

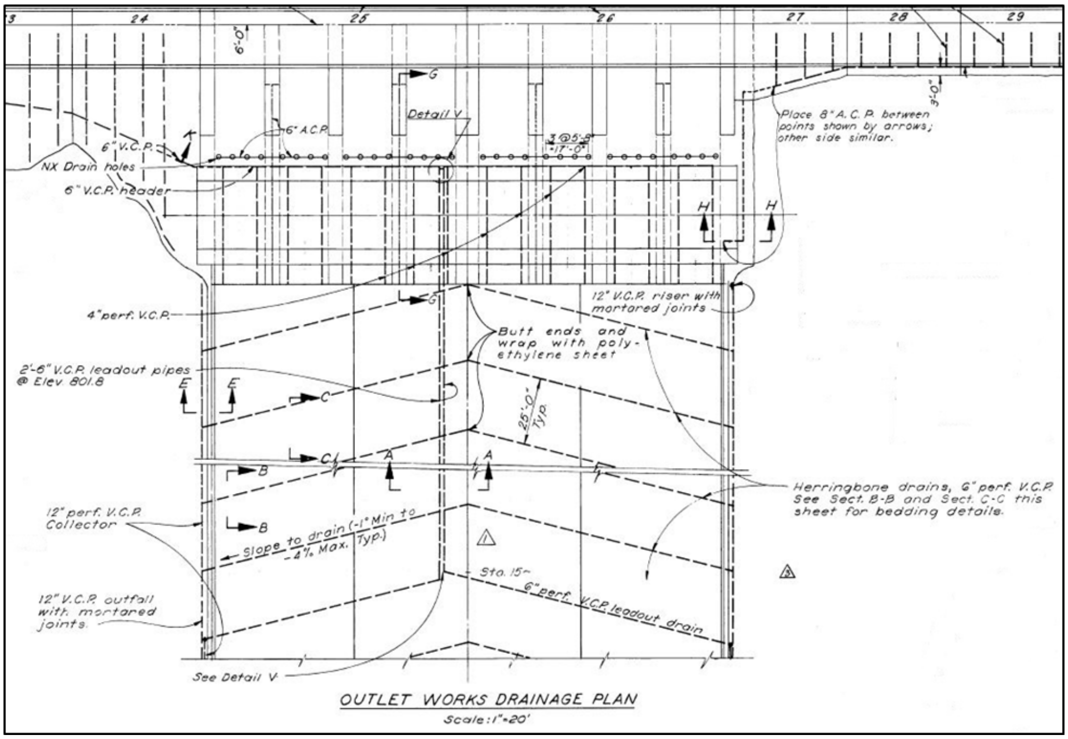


**Figure 3-4: Plan view of headworks (top) and upper spillway chute showing herringbone and collector drains (from Drawing A-3B5-3) [5]**

To connect the drains to the foundation when overexcavation occurred or was required, formed vertical gravel drains and gravel filled sonotubes were used, as discussed in Appendix A. To prevent contamination of the gravel during concrete placement for the chute slab sections, the drains were covered with plastic (polyethylene) sheets, as shown in Figures 3-5 and 3-6. Although some locations without plastic sheeting were found in the post-incident forensic investigations, the IFT believes that the sheeting was used at most locations.

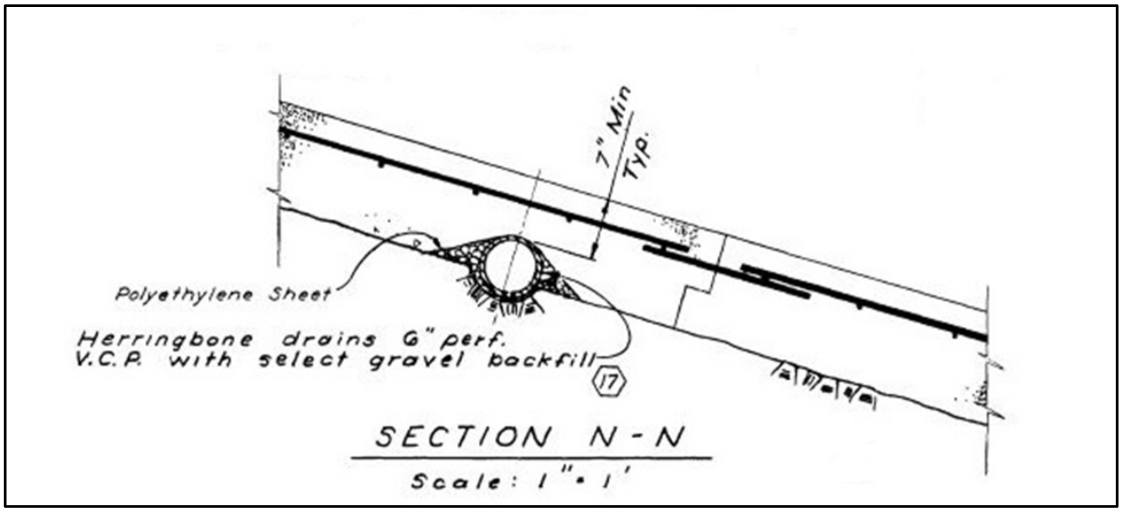


**Figure 3-5: Herringbone drain VCP pipe protruding into slab section [5]**

**Figure 3-6: Plastic sheets covering herringbone drains before concrete slab placement – Photo dated 09/30/66 (Photo OD_13202_09_30_1966) [8]**

The design indicated that the VCP herringbone drains were to have four rows of perforations in the lower half of the pipe. During the post-incident forensic investigation (see Appendix D), instances were found where some herringbone drain sections were constructed with the perforations in the top of the pipe and other sections of pipes were not perforated.

### 3.3.3 Chute Slab Reinforcement and Anchors

As-designed and as constructed, the chute slab included a single layer of reinforcing bars near the top of the slab, as shown in Figure 3-7. The reinforcing bars were No. 5 bars, spaced at 12 inches each way.

The chute design also included No. 11 anchor bars, spaced at 10 feet each way in plan view to extend 5 feet into the foundation. The anchor bar detail is shown in Figure 3-7.

### 3.3.4 Chute Slab Joints

The design drawings show that the spillway chute was intended to be constructed in 10 lanes, with the outer two lanes being the bases of the left and right chute sidewalls. However, the chute was actually constructed in six lanes, with the outer two lanes being the bases of the left and right chute walls. This change was reflected on some sheets in the as-built drawings, but not on others. In this report, the chute lanes are designated 1 through 6 from right to left, with Lanes 1 and 6 being the chute wall bases and Lanes 2 through 5 being the chute slab lanes, each 40 feet wide.

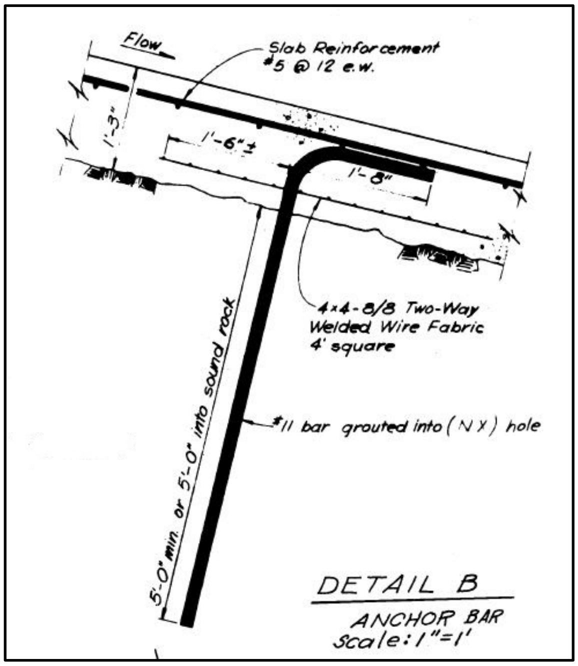


**Figure 3-7: Typical anchor bar installation [5]**

As discussed in Appendix A, the details on the design drawings and as-built drawings create considerable confusion concerning the types of joints used in the spillway chute slab. The discussion presented here is the IFT's best interpretation of the configurations of the chute slab joints, based on all available information.

The chute slab includes both formed and unformed transverse contraction joints. The as-built drawing details show formed transverse contraction joints including a 7½-inch deep key, in which the downstream edge of the upstream slab overlaps the upstream edge of the downstream slab, as shown in Figure 3-8. The detail also includes a ½-inch offset in which the top of the slab is lower on the downstream side of the joint, and transitions back to the normal chute slope within 6 inches of the joint. This detail has been observed in the field at some locations, but it is difficult to determine how frequently it was used, due to surface erosion of the concrete, which makes a ½-inch offset difficult to observe at some of the lateral joints. At some locations on the drawings, the lateral contraction joints were depicted without these offsets.

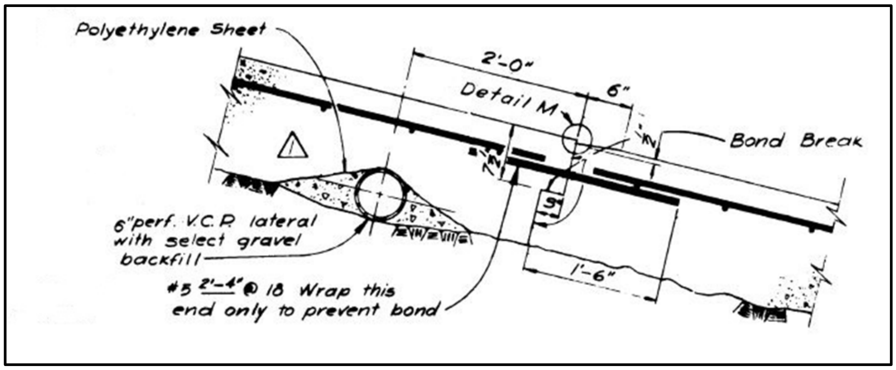


**Figure 3-8: Formed lateral contraction joint [5]**

Construction records and photographs indicate that much of the chute slab was constructed using slip-form methods, as shown in Figure 3-9. It is believed that the slip-forming used to construct the chute concrete allowed the contractor to place more than one 50-foot long concrete chute panel at a time. It is further believed that the contractor created unformed contraction joints by cutting shallow grooves in the freshly placed concrete to initiate a crack at intermediate design joint locations within the slip-form length, rather than stopping the slip-forming operation to construct a formed contraction joint. Formed contraction joints would have been constructed at the ends of the slip-form runs. According to the drawings, all contraction joints were to be treated with 1-inch deep by ¼-inch wide filler material, placed in a groove formed at the top of the joint. Corners of the joint were to be tooled to a round edge.

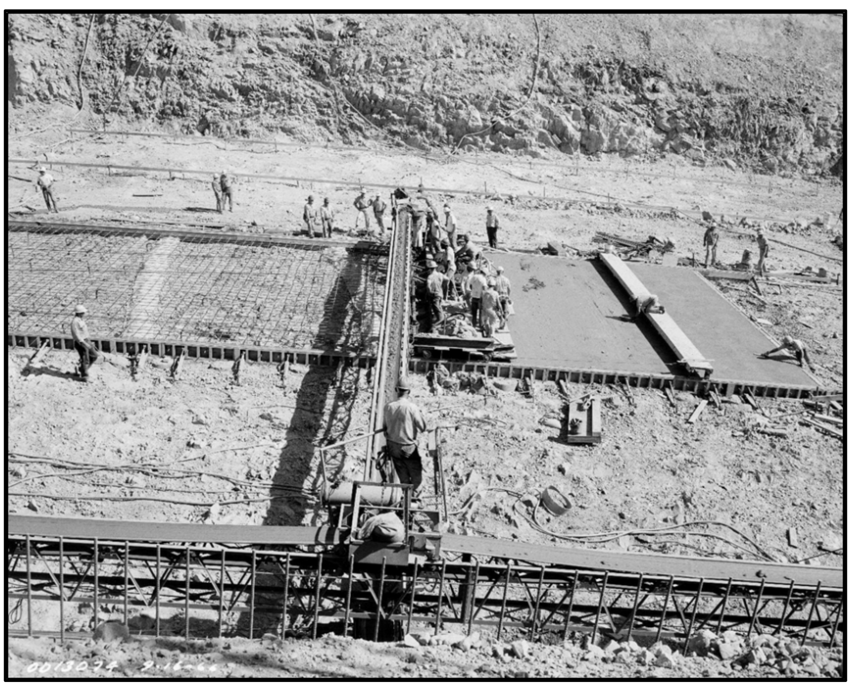

**Figure 3-9: Slip-form chute slab placement (Photo OD_13074_09_16_1966) [8]**

The drawings show that all longitudinal contraction joints were to be keyed. The specific key configurations vary for different longitudinal contraction joints, as discussed in Appendix A.

The drawings show No. 5 dowel bars at 18-inch spacing crossing each of the chute slab formed contraction joints (see Figure 3-8). These bars are embedded and bonded for a length of 18 inches on one side of the joint, and wrapped to prevent bond for 10 inches on the other side. These bars are shown to be placed just below the reinforcement mat at the top of the slab.

Although all formed joints were intended to be keyed, formed joints without keys were observed at a few locations during post-incident forensic investigations (see Appendix D).

### 3.3.5 Chute Slab Foundation

Based on the design drawings and specifications, as well as an interview, the design intent was for the chute slab to be founded directly on moderately weathered rock or better, which was to be pressure-washed to remove all mud, debris, and loose or unsound rock fragments. These specified foundation requirements would have resulted in a concrete slab that was well bonded to the foundation beneath, with anchors embedded into relatively competent material. However, the foundation requirements were substantially relaxed during construction. In some places, the prepared foundation appeared to have high points of reasonably sound rock surrounded by areas of what appears to be rock weathered to soil-like material, as shown in see Figure 3-10. These soil-like materials are described in the construction documentation as “compacted clayey fines.” In other locations, the prepared foundation appears to have been almost entirely rock that has weathered to soil-like material, as shown in see Figure 3-11.


**Figure 3-10: Example of the clean-up effort for the spillway chute prior to placement. Note what appears to be “compacted clayey fines” between the rock outcrops. (Photo OD_13484_11_10_1966) [8]**

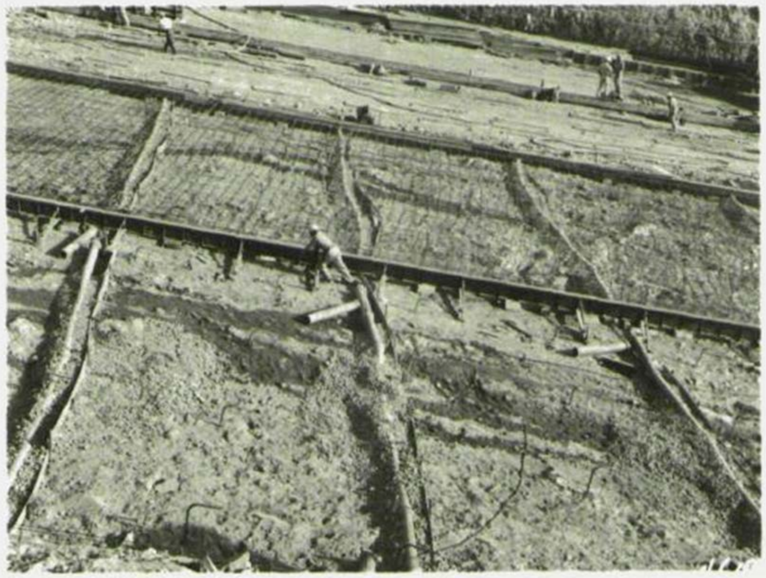

Photo 39. Chute foundation in vicinity of Sta. 33+60. Tile and gravel underdrains in lanes 2 and 3, rebar in lane 3. View southeast.
Neg. No. 4644 11-2-66

**Figure 3-11: Lane 3 downstream from Sta. 33+00 being prepared for placement. [9] Note what appears to be extensive areas of soil-like materials in the foundation.**

### 3.3.6 Chute Concrete

The chute slab concrete was specified to have a 28-day compressive strength of 3,000 psi, and all concrete cores tested in the post-incident field investigations had strengths higher than the specified strength. Type II cement was used for the concrete, and the coarse aggregate was up to 6-inch size. While large (6-inch) coarse aggregate can be beneficial in thicker placements, it would seem to be too large for a 15-inch slab with 3-inch reinforcing bar cover and 7-inch drain cover. The construction documentation also indicates that some of this aggregate caused breakage of the herringbone drain pipe as concrete was being placed. Although it is believed that broken drain pipes were repaired when they were observed, there may be other locations where pipes were damaged during concrete placement but the damage was not observed.

## 3.4 Spillway Operational History

The service spillway gates are operated to control reservoir levels in accordance with the operation plan for the facility [6].

The gates were first operated in 1969, within a year after the project was completed, with a maximum service spillway discharge of almost 82,000 cfs[5]. This was followed in 1970 by a peak discharge exceeding 110,000 cfs.

The history of daily maximum discharges for each year is shown in Figure 3-12. As shown in that figure, the service spillway operations vary from year to year, depending primarily on snowpack in the Sierra Nevada Mountains. The operating record includes extended periods of time when spillway gate operation was not required, e.g. 1975 through 1979, 1987 through 1992, and 2007 through 2010.

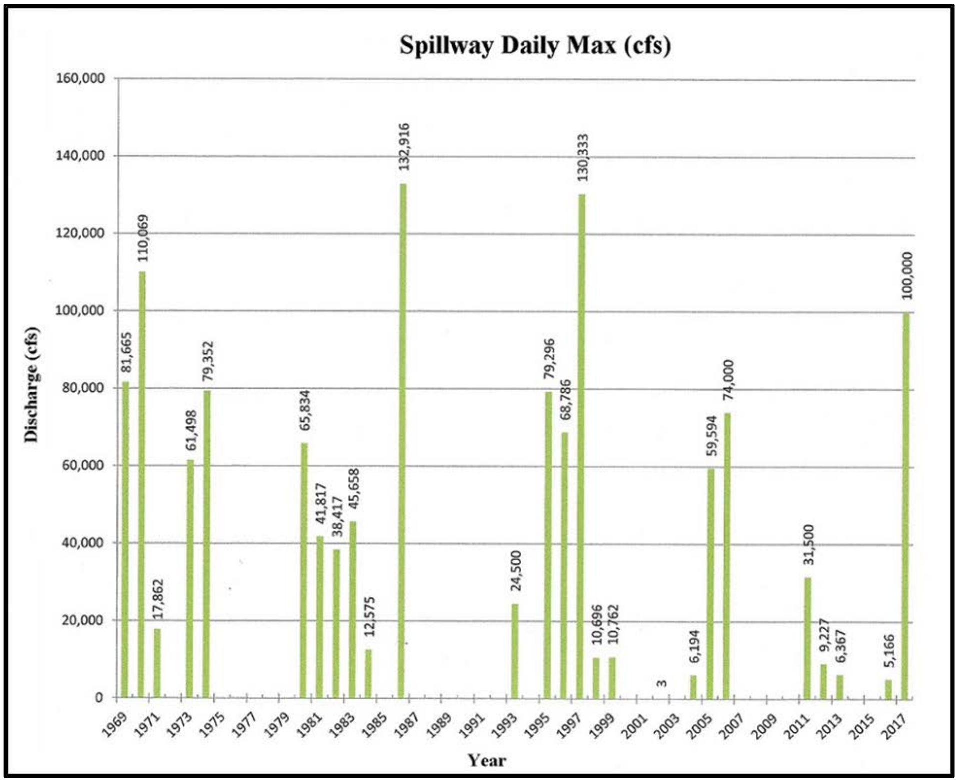


**Figure 3-12: Historic daily maximum flows (chart provided by DWR)**

A new record spillway discharge of about 137,000 cfs occurred in 1986, and then another new record discharge of about 160,000 cfs[6] occurred on January 1, 1997, which is still the record spillway discharge. As shown in Figure 3-12, the maximum daily average spillway discharges exceeded 50,000 cfs in eleven of the years during the time period from 1969 through 2016, most recently in 2006.

[5] The IFT notes that there are discrepancies among different tabulations of Orville Dam discharge values provided by DWR. Some tabulations include total discharges for the power plant and the service spillway, while others include only the service spillway. The IFT has used its best efforts to interpret the various discharge values, but all discharge values used in this appendix and this report should be considered approximate.

[6] Note that the spillway discharges presented in Figure 3-12 are maximum daily averages, and intraday discharges can exceed the daily averages as was the case in 1997.

## 3.5 Service Spillway Chute Slab Repairs

The service spillway chute slab concrete has been repaired numerous times since the original construction. Documented repair efforts occurred in 1977, 1985, 1997, 2009, and 2013. It is possible that there were other undocumented repair efforts. The chute slab repairs are discussed in more detail in Appendix G.

The observed spillway chute damage that led to the repairs was caused by various different mechanisms: cracking, removal of joint filler, delamination, and spalling. The cracking occurred predominantly over the herringbone drains. These cracks were first documented in a 1969 report [10]. This report indicates that cracks appeared above herringbone drains during slab concrete curing, within one month of placement. By the time of the 2017 incident, cracks were present over almost all the herringbone drains, and most cracks had likely been present since 1969. The cracks provided pathways for leakage through the slab at times when there was water in the chute.

Over time, the joint filler was removed from joints by the high velocity flows in the spillway. Loss of the filler would have resulted in increased leakage through joints when water was flowing in the chute.

Spalling occurred almost entirely in delaminated concrete at joints and cracks and at patches from previous repairs, generally in the concrete above the level of the reinforcing steel and dowels (see Figures 3-14 and 3-15). Delamination led to spalling of the slabs near the joints and cracks. The spalling exposed reinforcing steel in the chute slabs and dowel bars at the joints to corrosion, leading to bar failure (see Figure 3-13) when subjected to tension during the winter contraction cycle. Depending on the magnitude and orientation of the spalls, vertical faces that create stagnation pressures[7] could have been created.

The chute repairs have generally been shallow repairs that involved removing spalled concrete and patching the slab to restore the flow surface. In some cases, the repairs have also involved replacing the joint filler. In at least one of the repair efforts, the dowels in some of the formed lateral joints were cut in order to place joint filler material. Repairs have also included grouting of cracks in the slab. Repairs completed in 2009 and 2013 included use of a bonding agent, but, as discussed in Appendices D and G, the bonding agent may not have been properly applied. Consequently, the bond between old and new concrete may have been compromised, reducing the effectiveness of the repairs.

Based on reports of repair efforts and interviews with DWR personnel, the IFT understands that spalling both recurred in previously patched areas and occurred in areas that had not exhibited spalling before.

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[7] Stagnation pressure develops when flowing water hits an object in its path and kinetic energy of flowing water is converted into added water pressure as the flow is slowed or stopped. At unsealed joints and cracks this pressure can be transferred to beneath the slab.

**Figure 3-13: Concrete spall before repair [11]**

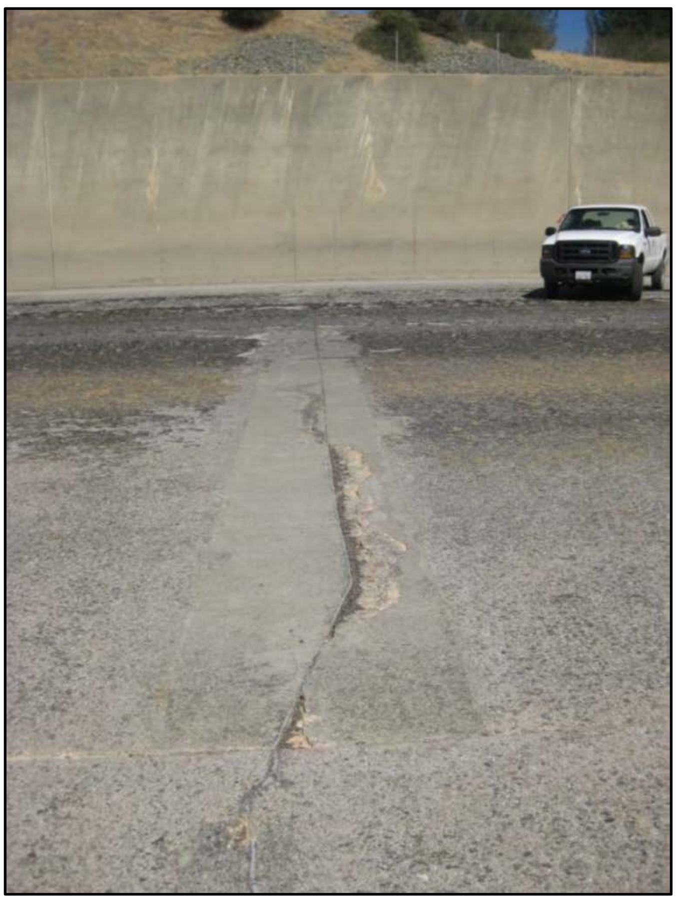

**Figure 3-14: Spalling of previous patch [11]**

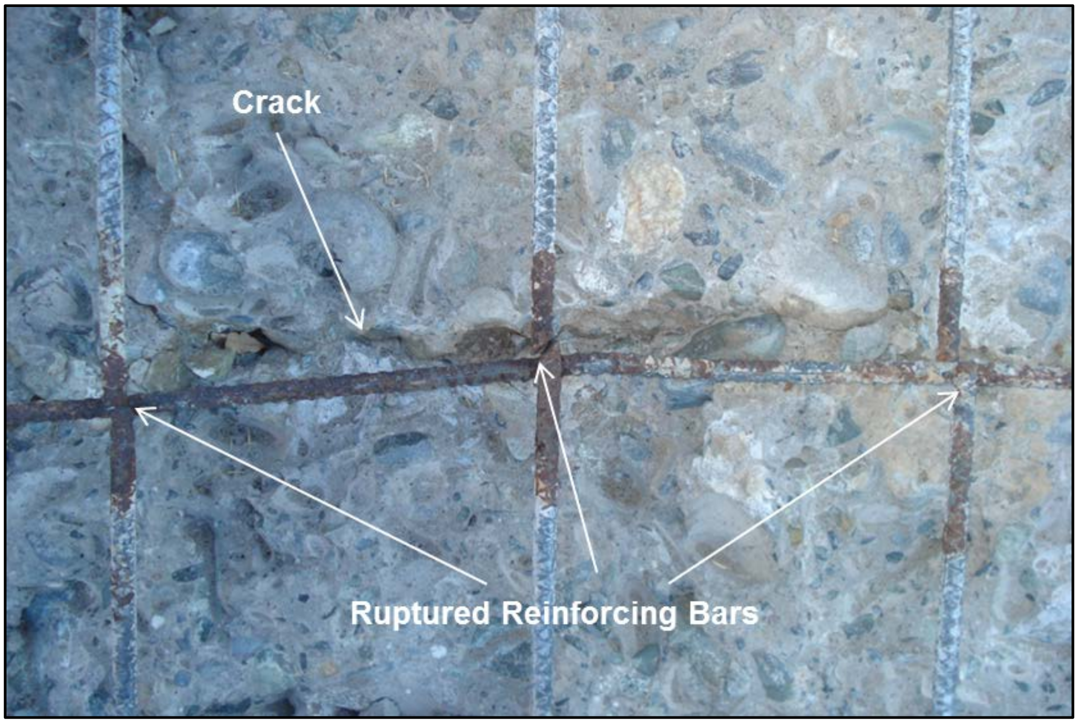


**Figure 3-15: Corroded and ruptured steel reinforcing bars at a slab crack [11]**

In the documentation of the 2009 repairs, the term “voids” was used. From discussions with DWR personnel involved in the 2009 repairs, the IFT understands that the areas designated in the documentation as “voids” were actually areas of delamination within the slab and not open voids in areas beneath the slab. The IFT further understands that no areas of voids beneath the slabs were identified or addressed in the 2009 repairs or in any of the other repair efforts.

## 3.6 Service Spillway Chute Underdrain Flows

In its first report, the Board of Consultants (BOC) which was convened after the February 2017 incident to provide review and advice for the spillway recovery and repair program, commented that:

> “The amount of drain water flowing from the pipe discharge openings along the spillway training walls seems extraordinarily large.” [12]

The IFT concurs. The amount of water that discharged from the drains when there was flow in service spillway is unprecedented in the IFT’s experience (see Figure 3-16).

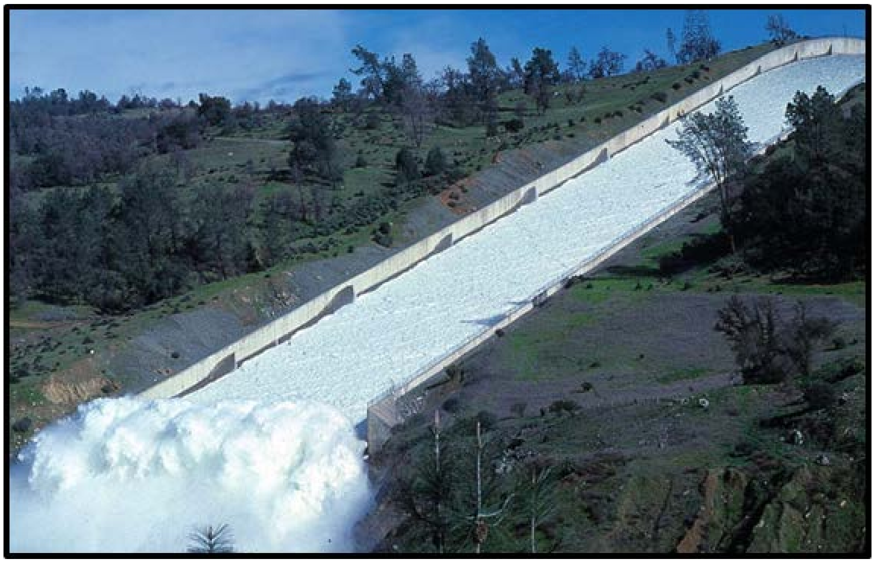

**Figure 3-16: Service spillway drain flows, February 3, 2006 (photo provided by DWR)**

In this investigation, the IFT has spent considerable effort in evaluating the service spillway outfall drain flows, as discussed in Appendix F1.

Significant flows from the service spillway outfalls were observed at the time of the first discharge through the spillway, in 1969, as described in DSOD notes and a DWR report [10, 13, and 14]. The same report states that the high flows from the outfall are related to spillway discharge and that the "high flows from the spillway drains are mystifying but probably not dangerous as the chute is anchored." The report also states that "inquiry to the designers regarding this aspect of performance would be in order." The IFT was not able to locate any documents indicating that the designer(s) were consulted at that time. From the design documents, the IFT is confident that the intent of the underdrain system was to collect seepage from groundwater beneath the slab, and not to collect large amounts of leakage from flows in the spillway chute.

Reports of flows from the spillway underdrain outfalls were sporadic and limited. From an evaluation of the available reports (see Appendix F1), there appear to have been several factors that affected drain flows when the gates were not discharging, and there were no clear patterns of how and why drain flows vary under these conditions. However, when the spillway was discharging, high drain flows consistently occurred, and the data indicate that the drain flows increased with increasing chute flow, though not necessarily proportionally.

The IFT agrees with the initial 1969 conclusion that the high drain flows are principally attributable to leakage through cracks and joints in the service spillway chute slab. This was dramatically demonstrated during post-incident repair work in 2017, when water was observed flowing on the spillway chute from leakage with the gates closed. To facilitate repair work, sand bags were placed on the spillway chute to direct the leakage flow toward a narrow area near the left chute wall. When the sand bags were placed, drain outfalls on the right side, which had been flowing significantly, stopped flowing.

High drain outfall flows are not limited to times when the spillway gates are open. Rather, significant outfall flows have been observed when the gates are closed and the reservoir level is high enough for water to be against the upstream sides of the gates. In these circumstances, gate leakage results in significant flow in the spillway chute.

Based on photographs and other reports, it has been noted that, at times, some of the drain outfalls did not flow while most of the other drain outfalls were flowing (see Appendix F1). From a review of the available information, the IFT has concluded that a particular drain may not have been flowing at one point in time, flowed at a later point in time, and then did not flow at a still later point, all under similar conditions. These variations in flow could have been due to partial obstructions of the drains, or they could have been an indication that there was less water injection in these areas, perhaps because of chute slab repairs.

In evaluating the drain outfall flows, it is important to recognize that, because of the design of the drain system, each outfall collects drainage from a series of herringbone drains located no closer than about 60 feet upstream of the outfall, as shown in Figure 3-17.

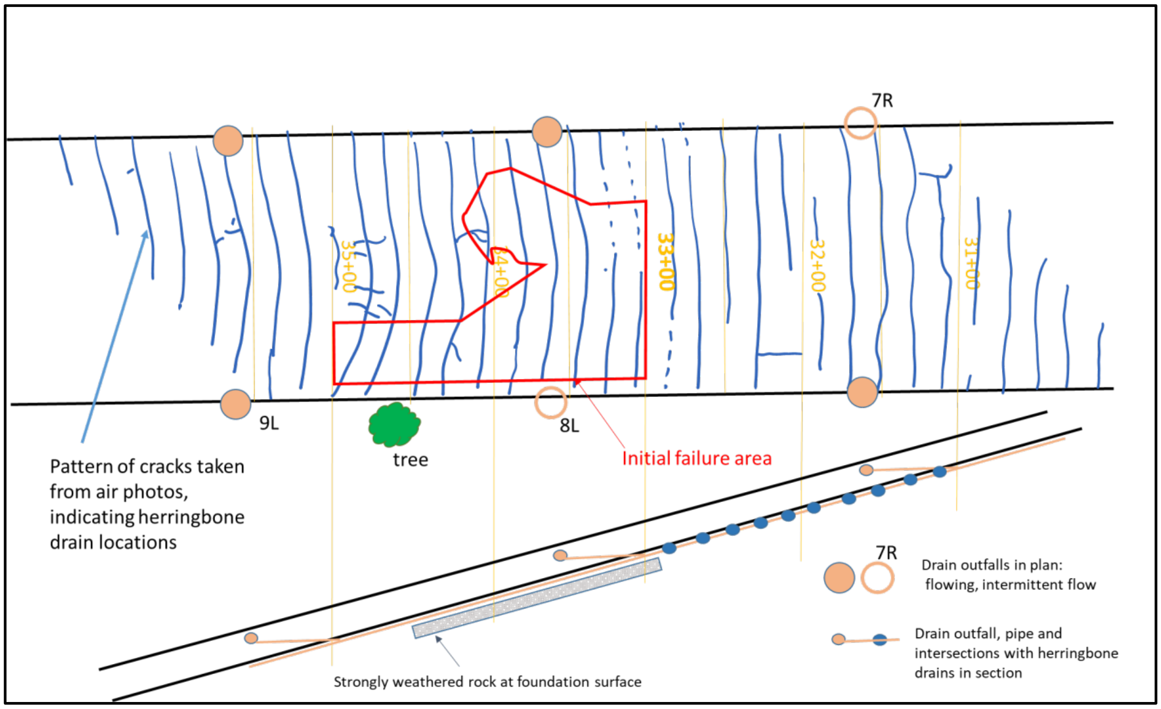


**Figure 3-17: Drain outfalls and herringbone drains in the vicinity of the initial chute failure**

## 3.7 Chronology of the February 2017 Oroville Dam Spillway Incident

The chronology reported in this section describes events at Oroville Dam during the February 2017 incident, and during the month preceding the incident. This chronology is based on service spillway discharge data provided by DWR, accounts of the incident obtained in interviews with DWR personnel, and from incident command center notes [15].

In January and February 2017, the service spillway experienced its first significant discharges since 2011, when the maximum discharge was 31,500 cfs on March 20, 2011 [16]. There had been some discharges in 2012, 2013, and 2016, but all were less than 10,000 cfs.

In 2017, there were reportedly no spillway discharges from January 1 through 12, 2017. Starting midday on January 13, spillway discharge was ramped up to about 9,700 cfs and maintained at that level through the rest of the day. The discharge was then reduced to about 6,600 cfs and maintained at that level through the afternoon of January 18, at which time the discharge was reduced to about 1,370 cfs for several hours, then further reduced to about 1,170 cfs for several more hours. The discharge was increased to about 3,000 cfs at 3:00 am on January 19 and maintained at that level for several hours, after which it was reduced slowly in steps starting at about 8:00 am, January 19, until the gates were fully closed at about 12:00 pm, January 20.

The gates remained closed through 4:00 pm, January 30, after which the spillway discharge was ramped up in several steps ranging from about 7,000 to 15,000 cfs. From February 1, 2017 through the morning of February 3, 2017, service spillway discharges were generally about 15,000 cfs; then discharges were increased to about 25,000 cfs and maintained at that level until mid-day on February 6, 2017, at which time the discharges were increased to between 42,000 and 45,000 cfs and held in that range until the morning of February 7. Figure 3-18 illustrates the chronology of the incident from February 4 through 25.

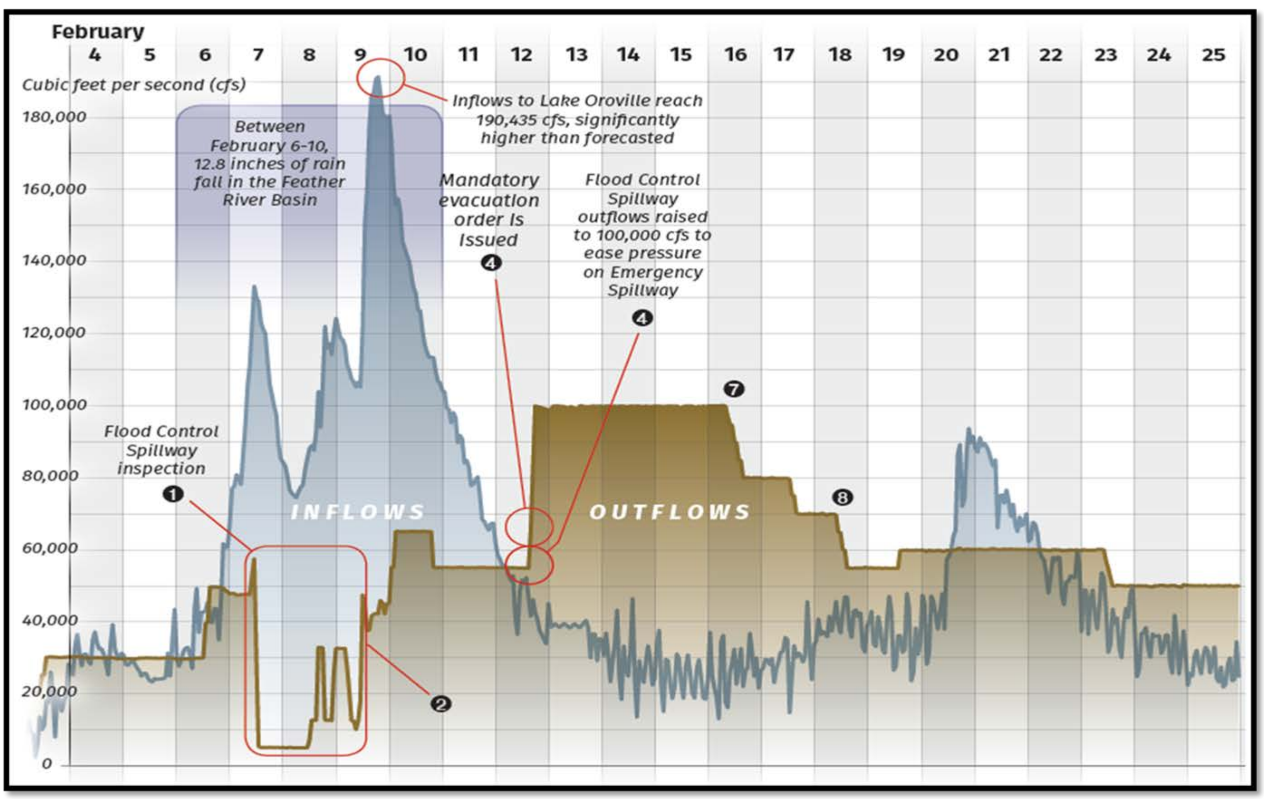


**Figure 3-18: Chronology of the February 2017 Oroville Dam spillway incident (from DWR)**

At about 10:00 am on February 7, 2017, service spillway discharges were increased again, starting at about 42,500 cfs, reaching about 52,500 cfs at about 10:20 am. Substantial disturbance in the service spillway chute flow was noticed by on-site DWR personnel at about 10:10 am on February 7, while the spillway discharge was being ramped up to 52,500 cfs.

After the observation of the disturbance in the chute flow, on-site DWR personnel contacted DWR headquarters in Sacramento, and an order to close the spillway gates was issued at about 11:15 am on February 7, 2017. Gate closure appears to have started at about 11:25 am, and the gates were fully closed by about 12:25 pm.

After the gates were closed, it was found that a significant section of the service spillway chute slab was missing, and a large erosion hole existed in the area where the slab sections were missing, as shown in Figure 3-19. This initial erosion hole at the service spillway was examined by a climb team on the morning of February 8, 2017.

**Figure 3-19: Service spillway chute damage observed after gates were closed on February 7 (from DWR)**

DWR knew that it would want to operate the damaged service spillway because of expected inflow to the reservoir, hence it was decided to begin opening the spillway gates to test service spillway capabilities in the damaged condition. The gates were reopened at about 4:00 pm on February 8, 2017, and, on February 8 through 10, DWR tried several test discharge rates ranging from 20,000 cfs to 65,000 cfs and monitored the associated progression of erosion at the service spillway. Spillway discharge reached 65,000 cfs at 3:00 am on February 10, and was held there for about 17 hours. At about 8:00 pm on February 10, the service spillway discharge was reduced to about 55,000 cfs and maintained at that level through 3:35 pm on February 12.

Meanwhile, inflows to the reservoir continued to increase due to a rainfall event, which was a major event, but not the largest in the history of the project. Sometime between about 7:00 and 8:00 am on February 11, the reservoir level exceeded Elevation 901, and water flowed over the emergency spillway crest structure for the first time in the facility's history. The reservoir level increased to a maximum level of about Elevation 902.6, about 1.6 feet above the emergency spillway crest, at about 3:00 pm on February 12, about 31 hours after the flow over the emergency spillway began. The flow over the emergency spillway at the peak reservoir level was estimated to be about 12,500 cfs. The emergency spillway discharge channelized as it flowed across the natural terrain downstream of the crest structure and caused extensive erosion, with some of the erosion areas headcutting aggressively toward the emergency spillway crest structure. According to Incident Command notes [15], at 3:44 pm on February 12, an evacuation order was issued for about 188,000 downstream residents, because of the rapidly advancing erosion areas in the emergency spillway discharge channel.

DWR opened the service spillway gates more, beginning at 3:35 pm on February 12, nine minutes before the evacuation order according to the Incident Command notes [15]. Service spillway discharge increased to about 100,000 cfs by about 7:00 pm on February 12. The 100,000 cfs service spillway discharge was maintained through 8:00 am on February 16. Discharge over the emergency spillway crest ceased at about 8:00 pm on February 12, about 36 hours after it began and about 5 hours after the flow had peaked.

At about 3:30 pm on February 14, the evacuation order was changed to an evacuation warning, under which residents were advised to monitor the media and be prepared to evacuate again, if necessary. No further evacuation orders were necessary, and the evacuation warning was lifted five weeks after the evacuation order was first issued.

DWR established a target reservoir level at Elevation 850, which is 50 feet below normal full pool level. Beginning February 16, service spillway discharges were adjusted based on estimated inflows to reach the target reservoir level. At 3:00 pm on February 20, the reservoir level reached about Elevation 849, and it was held at about Elevation 850 for the remainder of the month of February, through spillway discharges ranging from 80,000 to 50,000 cfs between February 16 and 27.

At about 7:00 am on February 27, gate closure commenced, with the gates fully closed by about 1:00 pm the same day. On-site investigations to support remedial actions began at that time. After that time, investigations and remedial actions were interrupted occasionally for service spillway releases to manage the reservoir. The service spillway gates were closed for the season on May 19, 2017, so that construction of spillway repairs could begin.

During service spillway operations between February 8, 2017 and May 19, 2017, additional spillway chute slab sections were lost and the erosion at the service spillway enlarged significantly, as shown in Figure 3-20.

**Figure 3-20: Ultimate damage at the service spillway (from DWR)**

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# 4.0 THE PHYSICS OF WHAT HAPPENED

This Section 4 provides the IFT's findings regarding "what happened" during the incident, from the standpoint of the physical factors and mechanisms which were involved, and the resulting physical sequence of events. Section 4.1 addresses the service spillway, Section 4.2 addresses the emergency spillway, and Section 4.3 describes physical factors which the IFT judged to be unlikely and/or not significant contributors to the incident.

The broader question of *why* the incident happened is discussed in Sections 5 and 6, focusing on the role of judgments and decision-making of individuals and groups, as well as broader human factors which include organizational, regulatory, and industry aspects.

## 4.1 <u>Service Spillway</u>

### 4.1.1 Chute Failure Initiation and Sequence of Events

The IFT believes that the service spillway chute failure most likely initiated by the uplift and removal of a section of the chute slab in the vicinity of Sta. 33+50 at about 10:10 am on February 7, 2017. Once the initial section of the chute slab was removed, the underlying moderately to highly weathered rock and soil-like foundation material beneath the slab at this location was directly exposed to high velocity spillway flow. The high-velocity flow rapidly eroded the foundation materials at this location, removed additional chute slab sections in both upstream and downstream directions, and quickly created the erosion hole that was observed by 12:30 pm on February 7, as flows diminished following spillway gate closure (see Figure 4-1). These findings are based on eyewitness accounts, as well as photographic and videographic records.


**Figure 4-1: Spillway chute damage observed after gates were closed (from DWR)**

The time of the initial chute slab failure is relatively well defined. On the morning of February 7, two DWR personnel were assigned to perform maintenance on a siren near the left side of the service spillway chute. As they traveled to the site that morning, they recorded video of the spillway from the road on the opposite side of the river. At the time of that video, the flow pattern in the spillway appeared to be normal.

The DWR personnel arrived at the siren location at about 10:00 am on February 7. At about 10:10 am, they heard what they said sounded like an explosion or a loud bang, and they saw spray coming from the spillway chute. They turned to see a significant disturbance in the spillway flow. At 10:23 am one of them began to record video and take photographs of the disturbed flow in the chute, as shown in Figure 4-2. They stated that the flow disturbance subsequently moved upstream and toward them (toward the left side of the chute).

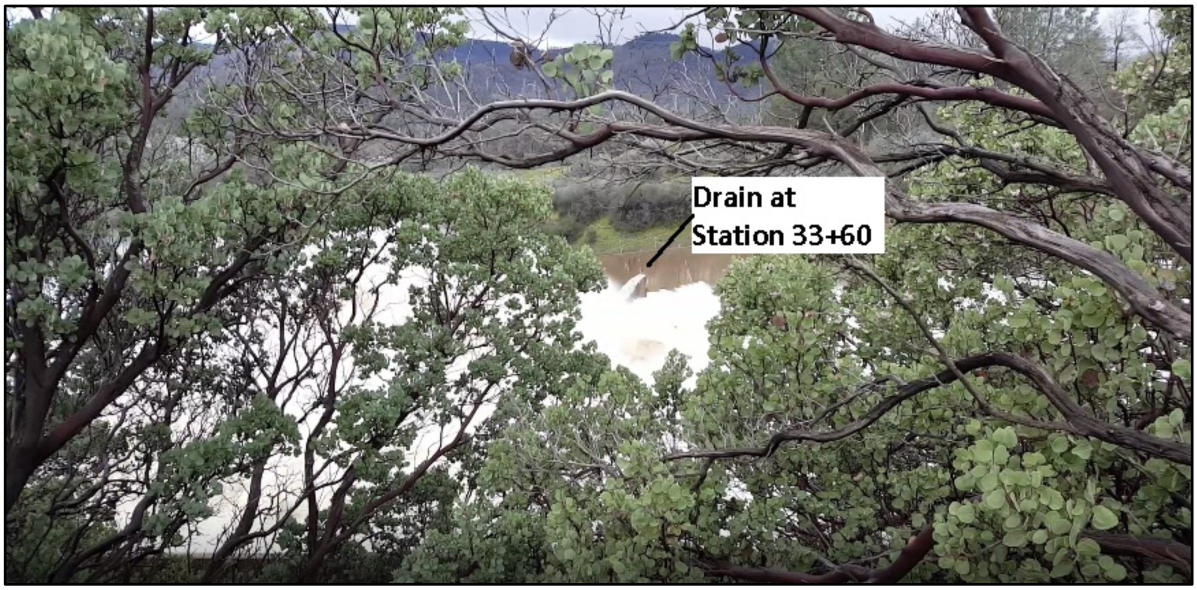


**Figure 4-2: Flow in the chute at 10:23 am (from DWR)**

Neither these two DWR personnel nor other eyewitnesses the IFT interviewed observed the initial failure, so it was not possible for the IFT to pinpoint the exact location of failure initiation. Based on a careful examination of the photographs and video recordings, it is the IFT's opinion that the chute damage most likely started in the vicinity of Sta. 33+50 in Lanes 3, 4, or 5. The disturbance in the spillway flow at 10:32 am, about 20 minutes after the initial chute failure, is shown in Figure 4-3. The initial damage is believed to be the result of a loss of a portion of the chute slab, and not an entire 40- by 50-foot chute slab panel. The initial slab failure could have been as small as a localized repair patch or a spall above a drain, or as large as a 20-foot section between the cracks above the herringbone drains. Once this initial portion of the slab failed, it likely rapidly triggered a chain of events, resulting in additional slab section failures.

The IFT cannot rule out initiation of the chute failure farther downstream, but the photographic evidence does not seem to support this interpretation. If damage occurred farther downstream, the damage quickly moved upstream of Sta. 34+00, as shown by the image from 10:32 am, in Figure 4-3. Such rapid progression is conceivable, if the removal of chute slab sections downstream of

Station 34+50 resulted in upstream slab sections sliding downstream, due to lack of downstream buttressing and poor foundation conditions.

The photograph in Figure 4-4 shows that the damage had progressed upstream to Station 33+00 by the time this image was recorded (about 12:07 pm).


**Figure 4-3: Flow in the chute at 10:32 am (from DWR)**


**Figure 4-4: Flow in chute at 12:07 am (from DWR)**

Since all physical evidence at the point where the failure initiated was destroyed, the exact location of the initiating event cannot be known with certainty. Others have suggested that the uplift of the slab initiated at Sta. 33+00, in part because of a disturbance in the flow pattern on the spillway seen in a photograph reportedly taken on January 27, 2017 (see Appendix I for more discussion). However, the IFT has concluded that, although there may have been water injection along the Sta. 33+00 joint, this was almost certainly not the location of the initial failure. The slab section downstream of the joint at that location could not have lifted without shearing the concrete key in the upstream slab section, and this condition was not observed in the photos taken by a climb team or in drone videos taken on February 8, which were carefully reviewed by the IFT. The intact, overlapping key element upstream of Sta. 33+00 can be seen in a drone video image in Figure 4-5. In addition, the photographic evidence and eyewitness accounts suggest that the initial damage occurred downstream of Sta. 33+00, and subsequently progressed upstream. The conditions at the Sta. 33+00 are discussed further in Appendix I.

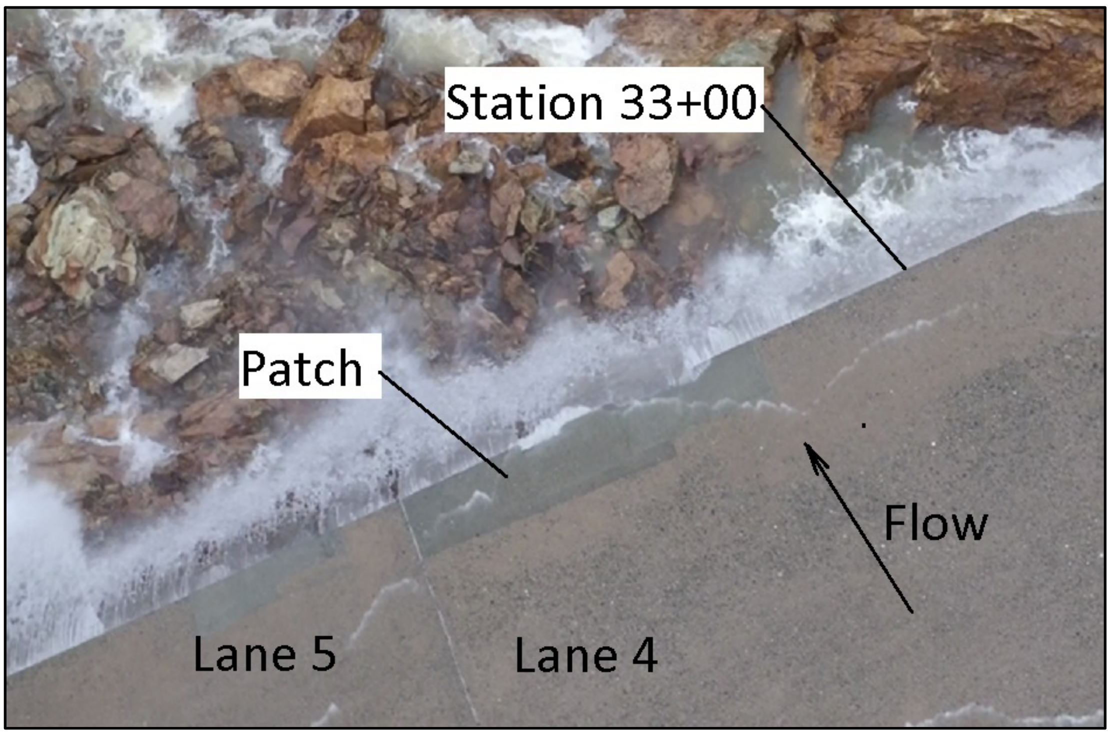


**Figure 4-5: Joint at Sta. 33+00 on February 8 (from DWR)**

Others have also suggested that the chute failure initiated when a section of the chute slab settled or “sagged” into a void beneath the slab. The void would have been created by piping (internal erosion) of foundation material from flow that entered the foundation through the slab and exited through the drain system. Although the IFT cannot absolutely rule out settlement or sagging of the slab, this scenario is believed to be less likely than uplift and jacking of a slab section for several reasons. First, the chute slab failure appears to have been very sudden, which, in the opinion of the IFT, is more consistent with uplift and jacking than with settlement or sagging. Second, the eyewitness accounts of a loud bang or explosion are also more consistent with uplift and jacking than with settlement or sagging. Third, although the IFT believes that voids may have formed beneath the slabs from piping (internal erosion), the IFT further believes that these voids would have been shallow, rather than deep. The strongly weathered rock foundation material likely was not entirely composed of erodible, fine-grained material, but also included coarser gravel-sized particles. As the finer material eroded, the remaining gravel particles would armor the surface and limit the depth of erosion. With shallow voids, the anchors would support the slab and reduce the likelihood of settlement. Fourth, the images of the spillway during times of low flow in January 2017 do not show flow disruptions that would be expected if a section of slab had settled or sagged. However, the IFT cannot rule out the possibility that minor localized settlement upstream of a crack or joint caused an offset into the flow that increased stagnation pressures and injection of water under the slab at that location.

### 4.1.2 Physical Factors Contributing to the Service Spillway Chute Failure

While there are numerous physical factors that contributed to the failure, which the IFT has studied in detail, the IFT has focused on the factors that it believes likely played the most significant roles in the failure of the service spillway chute.

The IFT has concluded that the initial uplift and removal of a slab section was most likely caused by water uplift pressure beneath a section of the chute slab, producing an uplift force that exceeded the uplift capacity of that particular section of slab. The resistance to uplift is provided by a combination of the weight of the slab, the weight of the water above the slab, the uplift resistance provided by the foundation anchor system, and any bond between the concrete slab and foundation. Once the upstream end of the slab section lifted, creating an offset into the flow, the pressure under the slab rapidly increased and produced the sudden failure of the slab.

As noted in Section 4.1.1, the initial chute failure section could have been quite small. Once even a small section of slab was removed, the high velocity flow would have quickly attacked the underlying erodible foundation material and begun to remove additional sections of the chute slab.

The excessive uplift pressure was mainly due to high-velocity spillway flow injecting water into and through slab surface features, such as open joints, unsealed cracks over the herringbone drains, spalled concrete at either a joint or drain location in either a new or previously repaired area, or some combination of these features. Localized slab deterioration and repairs existed in the area of the initial slab failure prior to February 7, and these localized areas would have been vulnerable to damage during high velocity spillway flow.

During spillway releases, the slab chute surface features allowed water to be injected into the foundation. The resulting underslab flow was observed at the collector drain outfalls and has been noted to be quite large. The IFT completed calculations of potential leakage through cracks and joints, as presented in Appendix B. Based on those calculations, the IFT believes that the flows through the cracks and joints could have easily exceeded the local drainage capacity of the underdrain system, resulting in the flow “backing up” around the drains. Water that collected around the herringbone drains would have increased the uplift under the slab.

Flows into the foundation would generally increase as flow velocities near the chute surface increased. The failure occurred very shortly after the spillway gates were opened more to increase discharge down the spillway chute, which led to higher surface flow velocities, and likely higher injection flows and uplift pressures.

In evaluating the physical factors, it is important to consider why the chute failure happened in 2017 at a spillway discharge of about 52,500 cfs, whereas the service spillway chute had not failed previously during higher discharges, most recently a discharge in excess of 70,000 cfs in 2006, and historically discharges up to as much as about 160,000 cfs in 1997. In other words, what changed from 2006 until 2017 such that the failure happened in 2017, rather than 2006 or earlier?

There are a number of possibilities for changes that could have occurred. The IFT believes that some combination of the following factors most likely was involved:

- New chute slab damage and/or deterioration of previous slab repairs
- Expansion of relatively shallow void(s) under the slab, through erosion or shrinkage of clay soils
- Corrosion of the steel reinforcing bars or dowels across the concrete cracks or joints
- Reduction in anchor capacity

Each of these factors is discussed briefly below.

**New chute slab damage and/or deterioration of previous slab repairs:** The service spillway chute slab has a history of repairs, as discussed in Section 3.5 and Appendix G. In the opinion of the IFT, these repairs did not successfully address the root causes of the observed deterioration, were not well suited to resist high velocity flows, and were generally short-lived, typically beginning to deteriorate with a year or two after completion. The post-incident forensic investigations showed locations of concrete delamination in the remaining chute sections, both in previously repaired areas and in new areas. New slab damage and deteriorated repairs resulted in more potential flow disturbance locations, potential for protrusions into the flow that create stagnation pressures, and more flow into the foundation, all of which could have made conditions on February 7, 2017 worse than during comparable or larger prior spillway discharge events. It can be seen in photos taken on January 13 and January 27, 2017 (See Appendix I) that drain flow appears to have significantly increased between the two dates. One possible explanation for the increased flow is that the number surface features in the spillway chute allowing flow to enter the foundation has increased between the two dates, because of spillway flows causing a number of previous repairs to fail, along with new damage to the chute. However, as discussed in Appendix F1, drain flows have been previously documented to vary significantly within a few days under apparently identical conditions, likely due to time lags following spill and rainfall events.

**Expansion of relatively shallow void(s) under the slab:** During construction, erodible materials were left in place in the foundation beneath the chute slab to varying extents and depths. Beneath the chute slab sections that were removed during post-incident forensic investigations, the erodible materials were found on top of and around areas of less erodible rock and are believed to be material left from incomplete cleaning and preparation of the rock foundation. In construction records, these materials were called "compacted clayey fines." Only one void of limited extent was found beneath the remaining slabs in the post-incident forensic investigations, despite investigation locations being targeted in part at areas with "compacted clayey fines." However, based on original geologic mapping of the chute foundation and limited available construction photos (see Figures 4-6 and 4-7), the IFT believes that erodible materials were much more prevalent in the area of the chute slab failure. With the large amount of water that has been observed flowing through the underdrain system and the lack of filter compatibility between the drain system gravel and the erodible materials, it is conceivable that voids beneath the slab could have been created by piping (internal erosion) of the underlying materials by water flowing under the slab. As noted in Section 4.1.1, the IFT believes that any voids that would have formed would have been shallow (perhaps a few inches at most), rather than deep. With extended periods of underdrain flows, areas of voids could expand laterally beneath the chute. This would increase the size of areas of uplift pressure beneath the slab and also more readily transmit stagnation pressures over greater distances, resulting in an increase in the net uplift force on the slab. With the thickness of the chute slab and the anchors which would support the chute slab, voids beneath the slab would be difficult to detect.

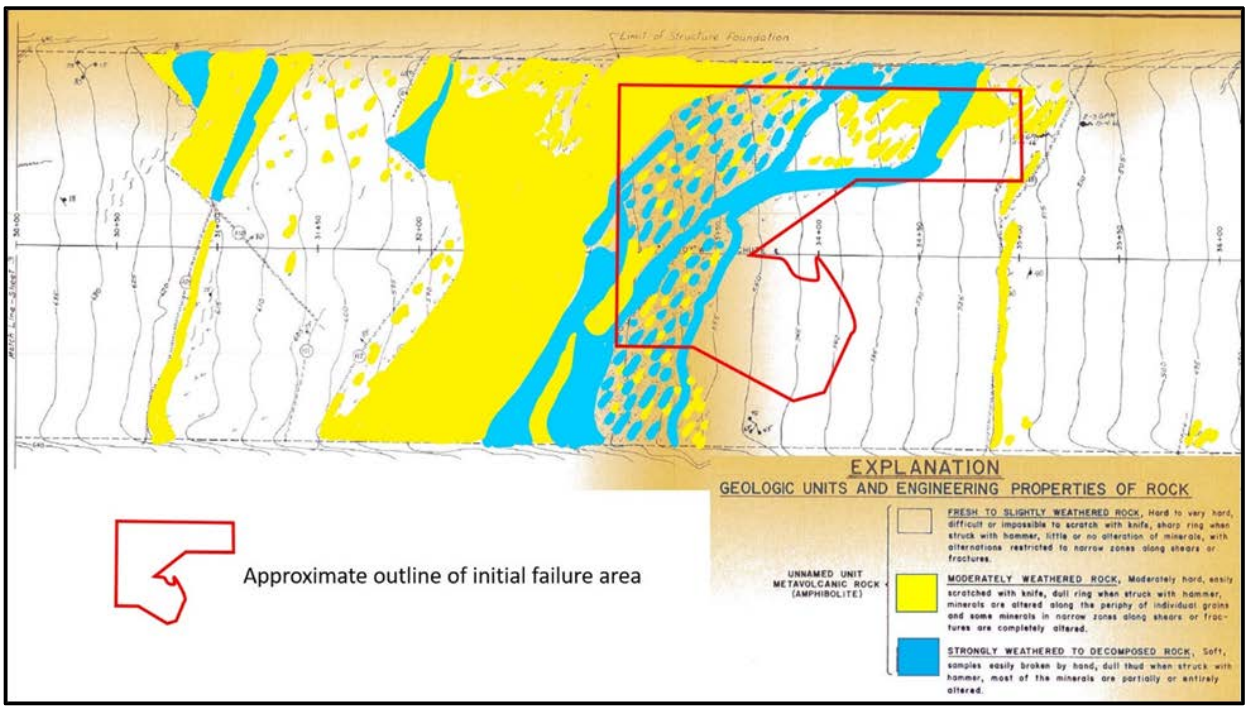


**Figure 4-6: Extents of moderately and strongly weathered rock. Geologic mapping from Final Geologic Report [9]**

**Corrosion of the steel reinforcing bars or dowels across the concrete cracks or joints:** Corrosion of reinforcing bars and dowels has been observed in the post-forensic investigations and in past slab repairs, as discussed in Section 3.5 and Appendices D and G. In some cases, ruptured steel reinforcing bars have been observed in slab cracks, likely the result of steel reinforcing bar sections reduced by corrosion and stresses caused by thermal contraction and expansion of the concrete slab. If the number of ruptured reinforcing bars increases over time, it reduces the uplift resistance of the associated sections of the slab.

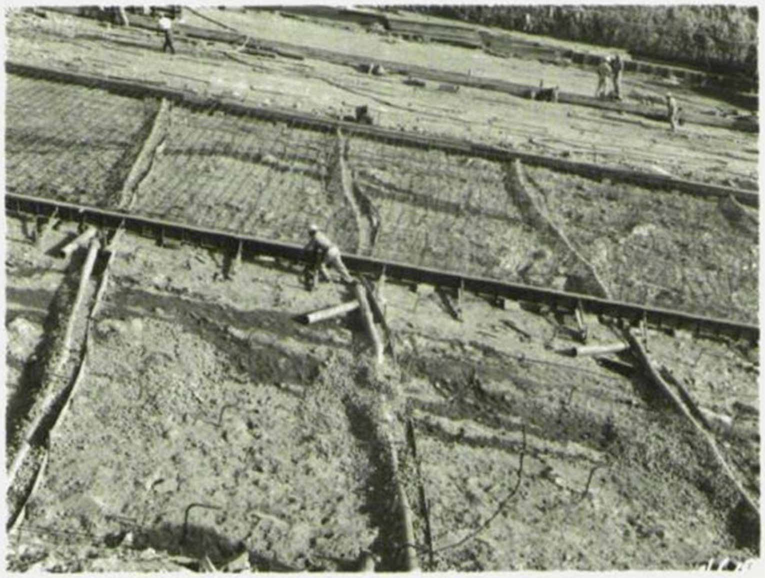

Photo 39. Chute foundation in vicinity of Sta. 33+60.
Tile and gravel underdrains in lanes 2 and 3, rebar in
lane 3. View southeast.
Neg. No. 4644 11-2-66

**Figure 4-7: Lane 3 downstream from Sta. 33+00 being prepared for placement. [9]**
**Note what appears to be extensive areas of soil-like materials in the foundation.**

**Reduction in anchor capacity:** Corrosion of slab anchors was observed in some of the post-incident investigations. Similar to the case for slab reinforcing bars, corrosion of anchors could reduce their cross sections and structural strengths. In addition, if piping (internal erosion) extended deeper at some anchor locations, it would have reduced the anchor strength by reducing embedment into the foundation.

The large flows in the chute underdrain system affect all four of the factors listed above. The IFT has studied the available information on outflows from the spillway chute underdrain system, as discussed in Section 3.6 and Appendix F1. It can be confidently concluded that, when water flows through the chute, flows observed exiting from the spillway chute drain outfalls are almost entirely from leakage through joints and cracks. The volumes of drain flow during spillway operation are much larger than from foundation drainage flows as intended in the design. Significant flows through the drains have been observed whenever there is flow in the chute, beginning with the first spillway discharge in 1969. Water has been observed flowing from the underdrain outfalls whenever the spillway gates are open to discharge water through the spillway. Underdrain outfall flows have also been observed many times when water is above the gate sill and the gates are closed, because the gates leak in this condition.

The foundation material beneath the chute slab in the area of the slab failure also plays a significant role in two of the four factors noted above. Based on the chute slab foundation geology maps and construction photographs (see Figures 4-6 through 4-8), as discussed in Sections 3.3.5 and Appendices A and C, the IFT believes that moderately to strongly weathered rock and soil-like foundation conditions existed beneath the slab in the area of the initial chute failure. These conditions reduced effectiveness of slab anchorage, allowed for at least some underslab erosion, and certainly dictated the extent of the ensuing damage after the chute failure initiated. Areas immediately downstream from the initial damage area have much less weathered foundation rock conditions, where both bonding of the concrete slab and bonding of the anchor bars to the foundation rock offered significantly greater resistance to uplift, and some sections of these slabs remained in place when the gates were closed after the initial chute failure.

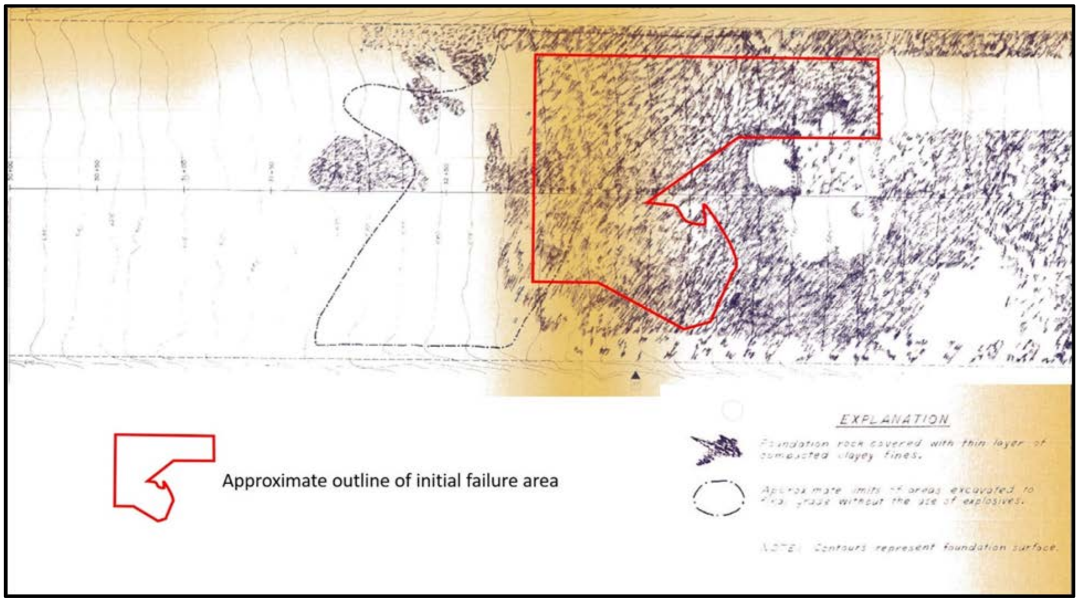


**Figure 4-8: Extents of compacted clayey fines. Geologic mapping from Final Geologic Report [9]**

The IFT has also identified a number of design and construction fragilities, specific to the chute slab details, which contributed to the chute slab's vulnerability to uplift. Specifically:

- Underdrains intruding into the chute slab section, reducing the thickness of concrete above the drains to 7 inches or less (compared to a design minimum slab thickness of 15 inches elsewhere). This condition resulted in cracks above most of the herringbone drains. The cracks allowed water to pass through the slab and also led to a propensity for concrete delamination and spalling in the slab.
- The formed contraction joints in the slab did not include waterstops, which facilitated water passing through the slab into the foundation. These formed joints were also too far apart to prevent wide cracks from opening over the drains.
- The specified foundation preparation and treatment, which was followed for the headworks and emergency spillway crest structures, was relaxed during construction of the spillway chute slab. Up to 50 percent or more of the foundation in some areas was not properly treated by removal of weathered materials and cleaning of soil-like materials from the surface, with no definition of the areal extent over which this 50 percent assessment was to be applied.

- The design included shallow rock anchorage, with only 5-foot embedment length into the foundation, and the embedment was possibly less where the slab was thicker than the minimum of 15 inches. In addition, some anchors in the initial chute failure area were installed in strongly weathered that was unlikely to develop the intended pull-out strength. The embedment of the anchors into the 15-inch thick slab was likely also inadequate to develop the full anchor bar strength.
- The underdrain system had numerous deficiencies, such as no filtering, possibly broken or disconnected pipes caused by the method of placement, incorrectly oriented or missing drain holes in the pipe, likely inadequate collector drain capacity for the flow that ultimately occurred through the slab, and a lack of redundancy.
- The slab had a single layer of nominal reinforcement bars near the top of the slab, rather than two layers of more robust reinforcing, which might have helped control cracking over the drains.
- With the joint dowels and steel reinforcing mat both being placed near the top surface of the slab, as compared to the dowels being placed at mid-height, there is an apparent plane of weakness created near the top surface of the joint, which could have increased the potential for delamination and spalling of the concrete near the joints.
- The maximum aggregate size in the concrete was relatively large (6-inch size), resulting in a propensity for cracking and spalling at keys and over drains, as well as damage to drain pipes during concrete placement.

## 4.2 Emergency Spillway

The development of the damage to the emergency spillway discharge channel was closely observed during the incident. Photographs and video footage, along with eyewitness reports, provide documentation of the development of the damage.

As noted in the description of the event chronology, flow over the emergency spillway occurred for about 36 hours on February 11 and 12. The flow peaked at about 3:00 PM on February 12, about 31 hours after it began, at an estimated discharge of about 12,500 cfs, which is only about 3 percent of the emergency spillway discharge that would be required to pass the Probable Maximum Flood (PMF).

During the incident response, in preparation for using the emergency spillway, trees had been cleared from the natural hillside downstream of the spillway crest structure before flow over the crest structure occurred. As the emergency spillway discharge flowed over the natural ground downstream of the crest structure, erosion began to occur. Erosion of surficial soil deposits was expected, however erosion began to develop to greater depths than expected. By the afternoon of February 12, concentrated areas of erosion were observed to be rapidly progressing upstream toward the emergency spillway crest structure, resulting in issuance of the evacuation order at about 3:44 PM on February 12, according to Incident Command notes. Figures 4-9 through 4-11 show the emergency spillway during the incident.

**Figure 4-9: Oroville Dam emergency spillway as overflow began on February 11 (from DWR)**

**Figure 4-10: Emergency spillway headcutting on February 12 (from DWR)**

The principal physical factor contributing to the damage at the emergency spillway was clearly the presence of significant depths of erodible soil and rock in features orientated to allowed rapid headcutting toward the crest control structure. The erodible materials appear to be associated with geologic features such as shear zones. Other factors that contributed to the damage at the emergency spillway include:

- Hillside topography that concentrated flows and increased erosive forces, facilitating headcut formation

- Insufficient energy dissipation at the base of the spillway ogee crest structure
- Absence of erosion protection downstream of the crest structures

**Figure 4-11: Emergency spillway after the incident (from DWR)**

The area downstream of the emergency spillway crest structure is for the most part a natural hillside with variations in topography. The hillside also contained some infrastructure features, such as access roads and transmission towers. The natural topographic variations and infrastructure created areas where the water flow would concentrate, increasing its erosive power. Some of the topographic low spots in the hillside likely corresponded with locations of more erodible materials, because natural erosion of these materials would have created low spots.

The ogee spillway crest structure has a relatively short toe apron, which would have provided only very limited energy dissipation before the water exited onto the natural terrain. Downstream of the crest structure toe apron, the hillside did not include any structures to provide erosion protection. If erosion had progressed to the short toe apron, the stability of the crest structure could have been compromised.

The IFT notes that the severe erosion observed at the Oroville Dam emergency spillway was not unprecedented. In fact there have been numerous cases of severe erosion of rock at unlined spillway channels. For example, in a 1990 report titled *Geotechnical Aspects of Rock Erosion in Emergency Spillways*, prepared for the US Army Corp of Engineers [17], it is noted that:

> "Experience has shown that severe erosion of rock and soils flooring unlined emergency spillway channels may cause undermining or failure of spillway structures and catastrophic release of [reservoir] water. Significant erosion-induced damage is well documented in spillway channels at projects built and managed by

> the US Army Corps of Engineers (USACE) and other Federal Agencies, and one large privately owned dam lost impoundment by spillway failure."

Regarding the mechanism of the erosion, based on an observational database of documented cases of spillway erosion, laboratory flume model studies, and computer simulation, the report notes that:

1. "The data base showed that severe erosion occurred at discharges which were less than 10 percent of Project Maximum Floods, and at velocities which were greater than those recommended by current guidelines; spillway channel erosion was driven by processes similar to knickpoint migration (headcutting) in natural stream channels, and the occurrence of stratigraphic and structural discontinuities in the spillway foundation were important factors in controlling the occurrence and extent of erosion."
2. "Furthermore, the model studies and computer simulations showed that erosion did not accompany peak discharge but rather it occurs on the lower portions of the rising and falling limbs of the hydrograph. These findings support the observation that severe erosion may occur at discharges significantly lower than the Project Maximum Flood or Spillway Design Flood."
3. "Severely eroded channels exhibited natural or constructed features which tended to concentrate flow and enhance erosion; these features included pilot channels, roads, ruts made by off-the-road vehicles, and topographic irregularities."
4. "… the potential for scour due to a backroller at the base of an overfall [at a headcut] diminishes … as the flow increases because the increasing momentum of the stream will cause the overfalling nappe to move away from the waterfall face."
5. "The rates of headcutting via this scour roller process can be quite rapid because the hydraulic forces are concentrated within a small zone situated at the toe of the waterfall. Although alternate scouring processes (i.e., tractive force scour) may increase with flow discharge, the rate of this type of scour may not be nearly as rapid as backroller-caused scour."

The erosion observed at the Oroville Dam emergency spillway was consistent with the findings of this 1990 report. As noted above, the emergency spillway discharge which caused the erosion was only about 3% of the spillway discharge capacity, the flow pattern was channelized by topography and infrastructure, the erosion was rapid and described by some observers as "shocking," and it was observed that headcutting was increased in locations where weaker materials were present in the rock.

## 4.3 Physical Factors Judged Unlikely and/or Not Significant

In a memorandum prepared in May 2017 [1], the IFT identified potential physical factors that it would be considering in its investigation: 24 factors related to the service spillway and four factors related to the emergency spillway. Most of the potential factors for the service spillway and all of the factors for the emergency spillway were found to be potential contributors to the incident and are discussed in Sections 4.2 and 4.3. However, the IFT found no significant evidence that five of the potential factors for the service spillway are likely to have been contributors, or found that they contributed only in a minor way when coupled with the other significant factors, as discussed in this section. In addition, during the course of the investigation, the IFT became aware of the potential for existence of an old mining adit in the vicinity of the service spillway, and this is discussed further below.

**Cavitation:** The conclusion that cavitation was not a significant factor is based on computations for historic flows and visual observations of the downstream chute which remained in place after the service spillway damage occurred. The IFT completed calculations related to cavitation potential, as presented in Appendix B, and concluded that, for the Oroville Dam service spillway chute, cavitation damage would be expected only downstream of Sta. 31+00, and there only for extended periods of discharge greater than 100,000 cfs. As discussed in Section 3.4, the spillway has not been operated for extended periods of time at discharges greater than 100,000 cfs, hence cavitation damage would not be expected. This conclusion is supported by visual inspection of downstream sections of the chute slab, in which telltale indicators of incipient cavitation were not observed.

**Groundwater flow:** The possibility was considered that significant flows of groundwater, either through the rock mass or via ungrouted exploratory boreholes, could have undermined the chute. Groundwater springs had been noted on the hillside during original design investigations [18 and 19], and these springs were presumably the reason for providing the spillway chute herringbone underdrains. There are also no indications in any records that the 1962 and 1964 boreholes were backfilled with grout, and one DSOD Inspection Report noted:

> "A small artesian flow (2 gpm) rises from an old exploration hole in the draw below the emergency spillway. The hole is reported to be 900 feet deep in good rock. The flow is clear." [14]

However, none of these boreholes are in the vicinity of the initial slab failure.

There is no evidence that groundwater flows have increased from the small irregular seeps as originally noted on the hillside prior to construction. On February 15, 2017, a flowing spring was reported located about 250 feet from the right side of the chute at about Sta. 22+00. Flow was observed only after pronounced rainfall, and the spring did not appear to have any known relationship to spillway discharge or reservoir level.

It was reported to the IFT that no significant groundwater inflows were encountered during backfilling of the erosion hole at the service spillway during the 2018 repairs. This observation suggests small amounts of groundwater flow at the service spillway location, although it must be noted that the reservoir level during repairs was lower than that at the time of the service spillway chute failure.

Given the generally low hydraulic conductivity of bedrock units of the type, condition, and geologic structure of those in the service spillway foundation, it is reasonable to expect that groundwater flows would be low to moderate and probably skewed mostly into the low range. Locally higher flows could occur due to presence of occasional open joints, but these would be neither consistent nor prevalent. In shear zones and other areas with higher fracture density, it is probable that weathering products would have reduced fracture porosity.

Therefore, although groundwater flows may have contributed in a very small way to uplift pressures that developed under the slabs, it is the IFT's opinion that they were not a significant contributor when compared to water pressure injected through slab joints and cracks when the service spillway was operating.

**Seismic damage:** The IFT completed a review of seismic activity in the vicinity of Oroville Dam over the last 20 years, with a conclusion that no earthquakes stronger than M 4.0 have occurred within 100 miles of the site in that time period. No ground motions generated by earthquake events strong enough to conceivably cause damage to large civil structures have been recorded by instrumentation at the project within the last 20 years. Also, the service spillway chute slabs and its foundations are not as vulnerable to the effects of strong ground motions as are certain other facilities at the project, such as the FCO headworks, which have experienced no observable seismic damage. For all of these reasons, the IFT concludes that seismic damage was not a likely contributor to failure of the chute slab.

**Large variations in slab thickness**: The post-incident field investigations and the original construction records indicate that the slab thickness varied significantly at different locations. Slab thicknesses ranging from 14.5 to 81.6 inches were recorded in the post-incident investigations, at locations other than the herringbone drains. The slab was less than the nominal 15 inch design thickness at only one location. Construction records of volumes of concrete placement in different sections of the spillway support the observed wide variations in slab thickness. Slab sections thicker than the design value would have two balancing effects with respect to uplift resistance: an increase in slab weight and a decrease in anchor resistance (if the anchor length or position was not adjusted). It is the IFT's opinion that the variations in slab thickness away from drain locations were not a significant contributor to slab failure. However, as discussed in Section 4.1, the thin slab sections over the drains were a major contributor to the chute slab failure.

**Plugging drains by tree roots**: Others have suggested that plugging of the drains by tree roots was a significant factor in the service spillway chute failure. The IFT does not believe that this is the case. Video inspections of drains remaining after the chute slab failure found some roots intruding into the drains, but they were small roots which would not be capable of plugging the drains. DWR commissioned a study of trees and tree roots at the site by a professor at University of California at Davis. The report of that study has not yet been issued, but the IFT has been advised that one of the conclusions of the study is that the species of trees present at the site adjacent to the spillway would not be expected to have roots of sufficient size to plug the collector drain pipes extending to the locations of the pipes.

A report issued by others [20] suggested that roots from a single tree located on the left side of the spillway plugged Drain Outfall 8L (See Figure 4-12), which was observed to not be flowing just prior to the chute failure. The IFT believes that this is not plausible because of the physical arrangement of the drain system.

The particular tree in question was located very close to the left side of the spillway about 100 feet downstream from the initial chute failure area and Drain Outfall 8L. The layout of the drain system is illustrated in Figures 4-12 and 4-13. As illustrated in the figures, roots from this tree could not reasonably have blocked the collector pipe connected to Outfall 8L, which was the outfall observed to not be flowing. If roots from this tree blocked any outfall, it would have been Outfall 9L, which was observed to be flowing.

The potential for drain blockage by this tree is discussed in more detail in Appendix F1.

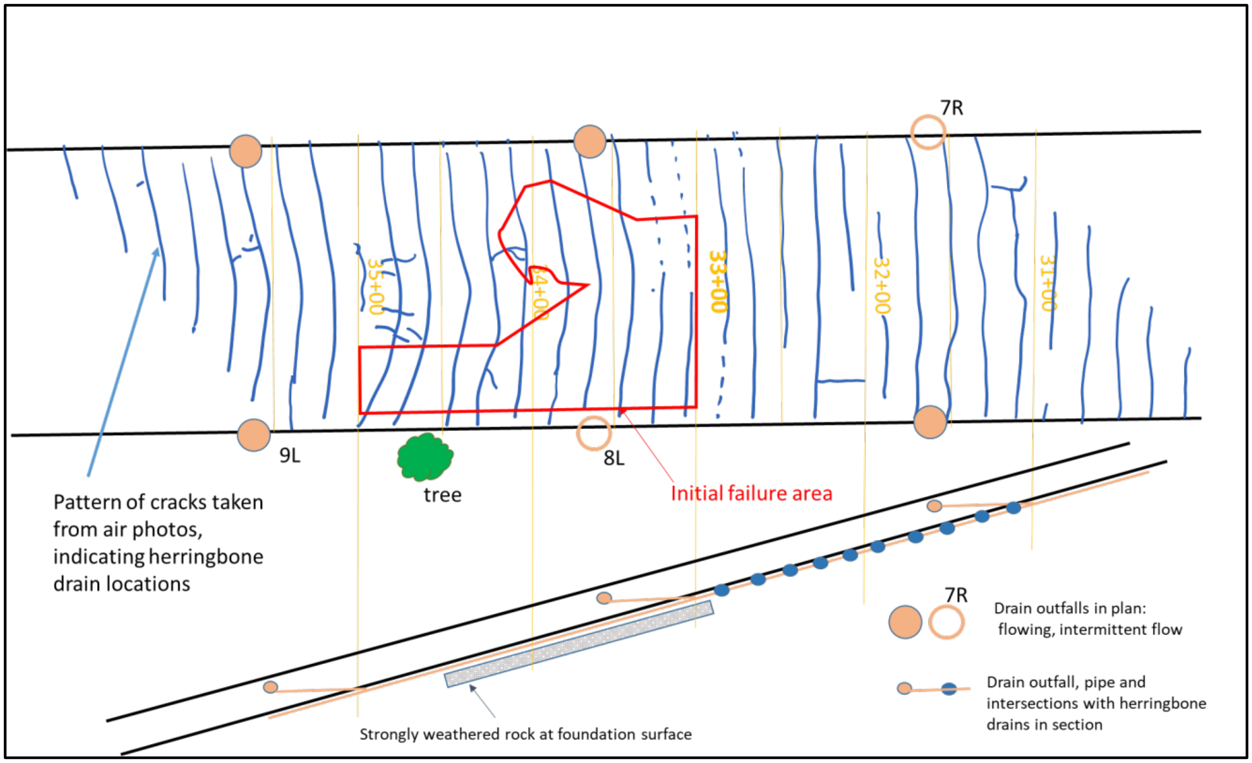


**Figure 4-12: Schematics of service spillway chute near initial failure area**

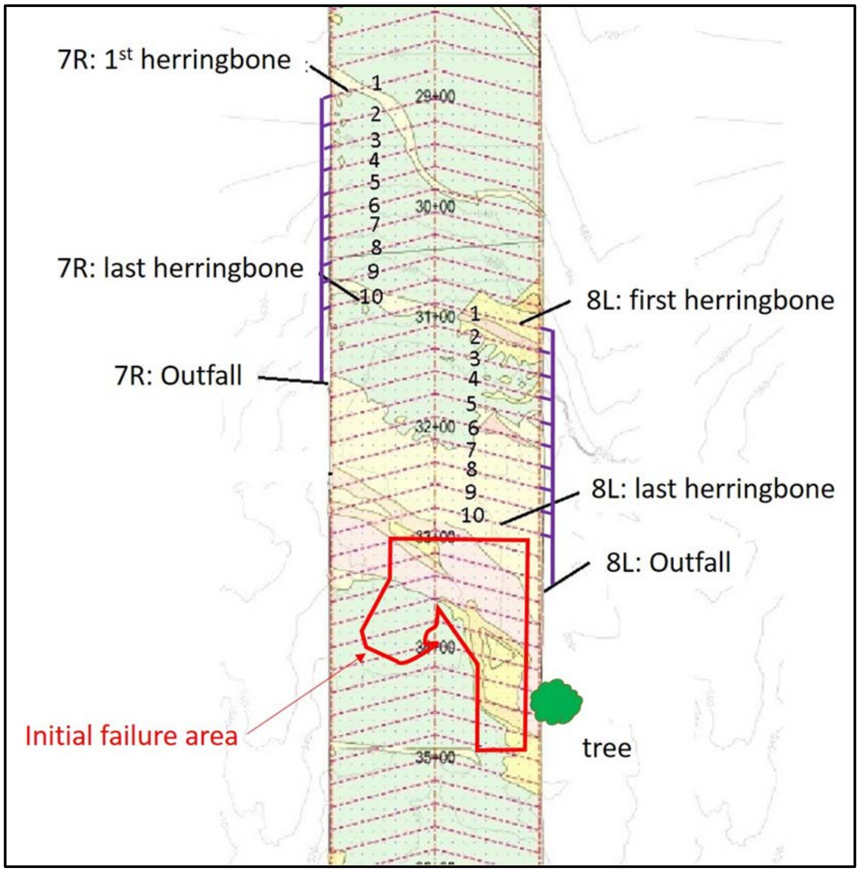


**Figure 4-13: Idealized layout of spillway chute underdrains upstream of initial failure area (underlay of chute geology provided by DWR)**

**Abandoned Adit**: The IFT was informed of the possibility of an abandoned mining adit underneath the service spillway. It was thought that the adit entrance may be an archaeological site that was originally recorded in 2003, as part of the cultural resources technical studies for the relicensing of the Oroville Dam. This site is located approximately 100 feet east of the spillway, between Sta. 22+00 and Sta. 26+00. However, following further investigation in 2017, the site was confirmed to be the remnants of a mid-20th century privy, possibly associated with the original Oroville Dam construction. An IFT search of archives at DWR and at California Department of Conservation (California Geological Survey and Office of Mine Reclamation) revealed no records of abandoned mining activities in the area of the service spillway.

A reference to an abandoned adit was later found in the 1962 Interim Report. It exists (or existed) at about Sta. 55+30 along the original service spillway alignment (i.e. in the hillside downstream of the current emergency spillway, quite near the Feather River) and, as such, would have had no influence on the service spillway chute failure.

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# 5.0 WHY THE INCIDENT HAPPENED

The February 2017 spillway incident was a result of interactions of numerous physical and human factors, beginning with the design of the project and continuing during the half-century until the incident. The factors specific to Oroville Dam and its spillways are discussed in this Section 5, with Section 5.1 addressing the service spillway and Section 5.2 addressing the emergency spillway.

Section 6 of this report separately discusses the more general organizational, regulatory, and industry factors which contributed to the incident. While these "general factors" contributed to the incident in a less direct way than the factors specific to Oroville Dam and its spillways, they did have a significant and important contextual influence in creating the long trajectory of events which led to the February 2017 incident.

Appendix J provides general information on the human factors framework and methodology which the IFT used for this investigation.

On the whole, the incident can be viewed as a "textbook" case of a major dam incident, in terms of typifying the extent of the contribution from human factors. The incident was preceded by decades of somewhat complex interactions and effects of human and physical factors, through which numerous warning signs of the impending spillway failure were missed, and many barriers, which were intended to provide "checks and balances," were overcome to eventually produce the spillway failure.

Overcoming so many barriers could be thought of as involving a degree of "bad luck," but more importantly, it indicates a long-term *systemic* failure of DWR, regulatory, and general industry practices to recognize and address the deficiencies and warning signs which preceded the incident. The incident cannot reasonably be "blamed" mainly on any one individual, group, or organization.

## 5.1 Service Spillway

The conditions that led to the service spillway chute failure have their roots in the original design and construction of the chute, which, combined with the geologic conditions at the site, resulted in vulnerabilities in the as-built structure. These vulnerabilities were not recognized in various inspections and evaluations that were completed throughout the history of the structure. In fact, warning signs of the vulnerabilities came to be accepted as "normal," and then generally not questioned further. Because the vulnerabilities were never recognized, chute slab repair efforts were neither well-conceived nor effective, and likely contributed to deterioration of conditions over time. All of these factors are discussed further below.

### 5.1.1 Design and Construction

The IFT compared the design of the Oroville Dam service spillway chute with information from 110 other spillway chutes designed in the time period from 1955 to 1975. This comparison is discussed in detail in Appendix E.

In general, the spillway chute design for Oroville Dam was found to be within the range of design practices of the time for spillway chutes on rock foundations, but toward the middle or somewhat below the middle of the range. One significant exception to this is the foundation drains protruding into the slab section. Only about 8 percent of the other designs reviewed had foundation drains protruding into the slab section, as is the case with the Oroville design, and none of them had drains comprising as large a percentage of the slab thickness as those in the Oroville design.

While the Oroville chute slab design was within the range of design practices for rock foundations – as apparently assumed by the principal spillway designer – it would not meet the typical practice of the day for spillway chutes on highly weathered or soil-like foundations. When conditions deviating from the design foundation conditions were encountered during construction, adjustments were not made to address these conditions.

Further, it is the IFT's opinion that, considering the height and slope of the Oroville Dam service spillway chute and the resulting hydraulic conditions, it would have been reasonable to expect the design to reflect the best practices of the day, which it did not. At the time of the Oroville design, chute slabs were being designed with substantially more robust features including two layers of reinforcing, more robust joint keys, foundation keys, and foundation drains fully beneath the slab and trenched into the foundation. Many of these features were incorporated into other spillway chute designs to address designers' concerns for leakage through the slabs during spillway discharge and the possibility of water uplift pressure beneath the slabs.

Spillway design practice today incorporates many of the more robust features listed in the previous paragraph and some of them have been refined and enhanced. In particular, waterstops in all chute joints became a common practice after the Oroville project was completed. In addition, the possibility of stagnation pressures developing if a vertical face (e.g. at a joint) extends into the impinging flow has been recognized and addressed in design details.

A contributing factor for the issues with the spillway design appears to have been the limited experience of the designer(s). An interviewee reported to the IFT that the principal spillway designer 1) was hired directly from a university post-graduate program, with his prior engineering employment limited to two summers with an engineering consulting firm, and 2) had no prior professional experience designing spillways, but had received instruction on spillway design in university coursework on hydraulic structures. The same source reported to the IFT that this engineer's design work was not overseen or directed by any engineer within DWR who was experienced in design of spillways. It was also reported that there no significant literature review or comparison with the designs of other large chute spillways built in the years and decades prior to the construction of Oroville Dam. This contradicts the opinion among many DWR staff that the entire SWP was designed by the "best of the best" (see Section 6.1.1 and Appendix K1).

Another contributing factor to the design vulnerabilities is that there was an apparent lack of communication between the designer(s) and geologists during the design. Reportedly, the principal spillway designer assumed that the chute slab would be founded on essentially good quality rock, except for limited areas where deeper excavation (overexcavation) and backfill concrete would be needed. However, the geologic information available at the time indicated that there were areas in the chute foundation where moderately to strongly weathered rock extended to tens of feet (see Appendix C). Hence, it appears that the amount of overexcavation that would be required to meet the design intent was not well understood by the spillway designer(s).

The vulnerabilities in the design were substantially exacerbated during construction (see Appendix A). There appears to have been little, if any, communication with the spillway designer(s) during construction. It was reported to the IFT that the principal spillway designer's only involvement during construction was a single visit to the site, during construction of the upper chute. It appears that the DWR construction team was making decisions regarding chute design and construction without any significant consultation with the principal spillway designer. The most significant decisions made during construction relate to the drain pipe size, the foundation preparation, and the lack of adjustments in anchor design in response to actual foundation conditions encountered (see Appendices A and D):

- **Drain pipe size:** The herringbone drain size was increased during construction from 4-inch diameter to 6-inch diameter, increasing the protrusion into the nominal slab thickness and increasing the propensity for cracking over the drains.
- **Foundation preparation:** Although the project specifications indicate that foundation for the service spillway, at both the headgate structure and the chute, was to consist of no worse than moderately weathered rock, prepared and cleaned by air and water jetting, these requirements were substantially relaxed for the service spillway chute. Specifically, the foundation requirements were relaxed to 50 percent moderately weathered rock or better, with no definition of the areal extent over which this 50 percent assessment was to be applied [21]. As a result, the chute slab foundation included substantial areas of varying thickness of "compacted clayey fines," which the IFT interprets to be soil-like weathered rock materials and rock fragments that were not completely cleaned from the foundation following the initial excavation to grade. In some areas of the foundation the "compacted clayey fines" appear to cover so much of the foundation that it would have been difficult to determine the nature of the rock below this material.

  Further, the IFT believes that in the area of the initial chute slab failure, the relaxation of foundation requirements led to construction of the chute slab on a relatively deep area of strongly weathered rock. The construction foundation maps show an extensive area of such materials beneath the slab, and the erosion that occurred rapidly during the initial chute slab failure suggests that this material extended to a significant depth.

  During construction, the conditions encountered in the area of the initial chute slab failure led to a suggestion to modify the design to include foundation keys [22], as would be used on soil foundations (see Appendices A and E). However, it appears that no changes were made.
- **Lack of adjustments in anchor length:** In several areas where strongly weathered rock were encountered, including the location of the initial erosion hole formed on February 7, 2017, it appears that anchor lengths or spacings were not revised to account for the weaker foundation (see appendix D)and in some areas the excavation depth was also not increased.

The lack of communication among geologists, designers, and construction personnel is not atypical of the 1960s dam construction era, however, it nevertheless substantially contributed to vulnerabilities incorporated into the original as-built service spillway chute.

The decision-making during design and construction may also reflect cost pressures, possibly combined with schedule pressures. One indicator of this is that the bid price for the spillway construction was reportedly about 10% below the engineer's estimate. It would not be atypical if the engineer's estimate served as an "anchor" relative to which the actual construction cost was judged, thus creating pressure to control escalation of costs to within the engineer's estimate, and substantially influencing decision-making during construction. In addition, as discussed above, the design team expected there to be a good rock foundation upon excavation to grade, and did not include a large bid quantity for overexcavation and backfill concrete, which further increased cost pressure. DWR was very concerned with the costs that were being incurred to achieve the specified foundation conditions, and this became a major point of contention between DWR and the contractor (refer to Appendix C). The Final Construction Report [23] indicates that there were very few places where the contractor was directed to overexcavate the foundation.

### 5.1.2 Inspections and Evaluations

Since the completion of the project, Oroville Dam and its spillways have been subject to many inspections and evaluations, including the following:

- Regular annual and semi-annual inspections by DWR, DSOD, and FERC
- Five-year Part 12D Independent Consultant and Director's Safety Review Board (DSRB) inspections/reviews, required by FERC and the State of California
- Potential Failure Mode Analyses (PFMAs), completed in 2004, 2009, and 2014

Through all of these efforts, the vulnerabilities of the chute slab to the failure that occurred in February 2017 were not identified by DWR or its consultants or regulators.

**Regular inspections:** Regular inspections have been performed by DWR, DSOD, and FERC throughout the history of Oroville Dam (see Appendix F2). However, in many of these inspections, the spillway chute was observed only from the deck of the spillway headgate structure. From that vantage point, only the upper, flat section of the chute can be observed. The spillway chute is difficult to access when water is above the spillway gate sill. The chute walls are tall and no access stairs or ladders existed. Further, when water is above the spillway gate sill, the gates typically leak, and DWR decided that the chute could not safely be inspected at close distance due to water on the gates, wetness of the chute surface, steepness of the chute, or other considerations.

The IFT understands that, in recent years, there may have been an option to work with in-house DWR safety engineers to develop alternative methods to safely perform inspections under those conditions, but these options were not exercised. Consequently, the chute slab surface was inspected close-up only when the reservoir was below the spillway gate sill and the chute was dry, and even then, the steeper section of the chute was not always inspected close-up.

However, had the chute been inspected close-up more often, it would have disclosed only the cracking and spalling, which, as discussed below, was not considered by inspectors to have been a serious problem. A contributing factor in this regard is that inspection checklists were generally not used during the inspections. Such checklists might have helped facilitate detection and evaluation of key failure modes for facilities such as the spillways. However, the checklists for the spillway would have needed to be developed by qualified engineers who understood spillway failure modes; otherwise, use of inadequate checklists could have instilled false confidence that the spillways had been adequately inspected and evaluated.

Flows from the spillway underdrains have been noted in inspection reports ever since the first operation of the service spillway in 1969 (see Section 3.6 and Appendix F1). At the time of the first observation of these large flows in 1969 [13], they were noted as being "mystifying" and were attributed to leakage through cracks and joints in the slab, rather than collection of groundwater, which had been the intended purpose of the underdrain system. Although questions concerning the drain flows and the cracking above the herringbone drains were raised in 1969, there is no indication that significant actions were taken in response. Subsequently, both the drain flows and the cracks above the drain came to be accepted as "normal," rather than being viewed as warning signs, and they were repeatedly reported in inspections with little or no concern expressed.

**Five-year inspections/reviews:** Numerous inspections and reviews of Oroville Dam, including its spillways, were completed at an approximately five-year interval starting in 1973, soon after construction was complete, and continuing through 2014 (see Appendix F2). The reviews performed in 2004, 2009, and 2014 were coordinated with the PFMAs performed in the same years, as discussed below. These five-year reviews were mandated by federal and state regulations, and were substantial efforts, which involved gathering and evaluating a large amount of information. However, they tended to focus on what had changed since the prior five-year review, and, with the exception of the 2014 effort, they did not identify or evaluate the spillway failure modes which occurred during the February 2017 incident. The spillway failure modes appear to have been "off the radar" during almost all of these five-year reviews.

**Potential Failure Mode Analyses (PFMAs):** In the PFMAs completed in 2004 and 2009, no potential failure modes (PFMs) of concern were identified for either of the spillways. In the 2014 PFMA [4], the PFM that initiated at the service spillway in February 2017 was essentially identified, but judged to be a "Category IV" PFM and ruled out, because it would not result in uncontrolled release of the reservoir (see Appendix F3).

The main reason for ruling out this PFM in 2014 was a judgment that the rock foundations were generally non-erodible. It appears that the conclusion regarding the foundation was heavily influenced by DWR opinions provided during the PFMA session. It further appears that the geologic records were not diligently reviewed to identify available boring logs and other information that belies the "myth" that seems to have developed over the years regarding the erosion resistance of the rock in the right abutment of the dam (see Appendices C and F3).

The team also concluded that the chute slab was in good condition. However, as reported to the IFT, the possibility of a chute failure was not investigated in detail, but rather the following hypothetical question was asked: If the chute slab failed, would erosion progress to fail the headgate structure and result in an uncontrolled release of the reservoir? The PFMA team's answer to this question was "no," based on its interpretation of the geology. Judging from performance of the spillway after the chute failure, this conclusion may have been correct. However, the February 2017 incident shows how, by limiting the PFMA process to considering mainly failures resulting in uncontrolled release of the reservoir, the process can miss component failures that can be very significant to the dam owner and downstream residents. The conclusion concerning erosion resistance of the rock may also have limited any further discussion of the early stages of development of the potential failure mode (e.g. chute slab failure), which could have linked issues with the chute to this potential failure mode.

The 2014 PFMA team's conclusion that the spillway chute was in good condition appears to have been the result of the team's observation of the chute surface, including repairs, some of which had been completed just a year previous, in 2013. The PFMA report makes no reference to the 2013 repairs, and the team may have incorrectly believed that the repairs they saw were completed in 2009, and therefore were proving to be durable. The relationship between the repairs and the protruding drains and the slab joint design and construction was also apparently not identified or explored.

In fact, the IFT can find no record that, in the history of operation of the project, there had ever been a detailed review of design, construction, and performance of the service spillway slab. Such a review could have identified the following warning signs and deficiencies:

- The relationship between the protruding drains and the cracks
- The relationship between leakage through the cracks and joints in the chute slab and the drain flows
- The failure of reinforcing steel bars at cracks
- The foundations that, in places, consisted of strongly weathered rock

Recognition of this collection of factors could have led to a conclusion that the chute slab was vulnerable to an uplift and slab jacking failure, followed by erosion of the foundation.

### 5.1.3 Chute Repairs

There have been five documented service spillway chute repair projects in 1977, 1985, 1997, 2009, and 2013 (see Section 3.6 and Appendix G). Available documentation of the first three repair projects is limited, while the documentation is more complete for the 2009 and 2013 repairs. However, the documentation of these latter two repair projects would not be considered thorough and detailed.

The repairs have addressed spalling, delamination, and cracking. In the IFT's opinion, the repairs have generally been limited in scope, and have lacked sufficient robustness to be long lasting. It appears that the relationship of the repairs to the inherent vulnerabilities of the chute slab and its foundation and the potential failure mode that initiated in 2017 was not recognized, but rather the repairs were judged as "routine maintenance" to maintain the chute slab grade. It was not recognized that repairs which deteriorated in as little as few years after completion posed risks of increasing leakage into the foundations, increasing uplift pressures, and possibly creating vertical projections into the flow at spalls, which could result in stagnation pressures that could greatly increase slab uplift pressures. It also appears that it was not understood that development of deterioration of concrete in new areas, not just in previously repaired areas, was an indication of likely widespread delamination of the concrete and likely continued degradation of the condition of the slab. In some of the repair projects, attempts were made to fill joints and cracks, but these repairs could not adequately resist thermal stresses and/or high velocity flows, and were, therefore, very short lived.

The 2009 chute repairs were designed by DOE and constructed by a contractor. The repair areas included in the specifications were in part identified by a “chain drag” survey to identify “drummy sounding” areas indicating possible delamination within the concrete. In the IFT’s opinion this method may have identified some areas of delamination, but it could not be expected to identify all such areas. The repair design was performed and reviewed by structural engineers with general experience with concrete repairs, but not with repairs of hydraulic structures with high velocity flows. DOE did retain an experienced concrete technology consultant to provide input on the repairs, but the scope of this consultant’s services was limited to providing input on specifications for repair materials and methods, but did not include review of the repair design or participation during construction. Information provided to the IFT also indicates that the specified bonding agent used in the repairs was not applied according to the manufacturer’s recommendations, which could have compromised the bond between new and old concrete. These repairs started to exhibit distress relatively soon after they were completed, and DWR tried unsuccessfully to pursue remediation of this distress by the contractor on a warranty basis.

DWR’s Oroville Field Division (OFD) completed another set of chute repairs in 2013, reportedly (according to interviews) using the specifications from the 2009 repairs. OFD’s maintenance workers completed this work, with limited or no involvement of O&M headquarters or DOE. Although the 2009 repair specifications were apparently used, interviews indicated that the 2013 repairs were completed without the same level of understanding and attention as the 2009 repairs, and may have been “rushed.”

The chute repairs which DWR performed in 2009 and 2013 were viewed as “routine maintenance” and not submitted to the regulators for review and approval. This was in line with the general United States practice of regulators to treat such chute repairs as routine maintenance, as was the case with DSOD, FERC, and the Oroville service spillway chute repairs.

## 5.2 Emergency Spillway

Two factors that contribute to why the emergency spillway damage occurred are misunderstanding of the geology of the right abutment and the decisions made during the incident that allowed flow over the emergency spillway crest to occur. Both of these factors are discussed below.

### 5.2.1 Geology

Prior to the February 2017 spillway incident, the geology of the right abutment of the dam, including the hillside downstream of the emergency spillway crest structures was fundamentally mischaracterized and misunderstood by DWR, its consultants, DSOD, and FERC. The history of geological evaluations of the right abutment is discussed in detail in Appendix C.

Pre-design and design geological explorations in 1948 and 1961 correctly recognized the characteristics of the rock in the right abutment, including the existence of areas of very weathered rock extended to depths of tens of feet. A 1962 geology report [18] fully described the typical deep weathering pattern in bedrock, and clearly recognized its very irregular pattern, noting that "weathered rock will of course be subject to relatively accelerated erosion; where this is critical, the rock should be protected."

As noted previously, it was reported to the IFT that there was little, if any, communication between the geologists and the spillway designer(s) during the original design of the project. The emergency spillway was designed with the intention that the crest structure would be founded on "good" rock, and that the downstream hillside would be left in its natural condition. The thinking was that, if the emergency spillway operated, erosion would occur on the hillside, but the crest structure would not be threatened, and the reservoir would be retained. It appears that the expected erosion of the hillside was neither detailed nor quantified in the design documentation. In the IFT's experience, this approach to emergency spillway design was not atypical at the time of the Oroville Dam design.

Detailed borehole logs containing the accurate descriptions of the moderately to highly weathered bedrock are given in the 1962 and 1964 [18 and 19] Interim Geological reports, but are omitted from the Final Geological Report [9]. The opening description in the Final Geological Report includes the following statement:

> "Foundation rock for the entire spillway is amphibolite, which contains numerous narrow shears and schistose zones. Fresh amphibolite is hard, dense, fine- to medium-grained, greenish gray to black, and generally massive, although a slight foliation (regional structure) is usually present."

Excerpted and read in isolation from the remainder of the report, this passage describes very favorable foundation conditions, and this depiction may be a major factor in the ensuing misinterpretations of bedrock conditions.

After construction it appears that DWR geologists developed and promulgated an incorrect understanding that the right abutment was composed of non-erodible rock, with a thin, 3 to 4 feet thick cover of soil. This interpretation is reflected in DWR memoranda in 2005 and 2009 and in the considerations of the 2014 PFMA team.

In the early to mid-2000s, as part of the Oroville Dam re-licensing process, external groups questioned [24] the safety of the emergency spillway regarding the potential for bedrock erosion, based on the original records and reports. DWR was requested by the FERC to investigate this issue, and a very brief review was undertaken (see Appendix C). The review resulted in a 2005 DWR memorandum [25] which stated:

> "The Emergency Spillway does not empty onto a bare dirt hillside. Instead, it empties onto a hillside composed of solid amphibolite bedrock extending from the spillway crest down to the Feather River. Where the rock is fresh, it is hard, dense, fine- to medium-grained, greenish-gray to black and generally massive. Even though this rock contains numerous narrow shears and schistose zones, variable weathering, joints and fractures, it is considered an excellent and competent spillway rock."

The memorandum was concluded with the following statement:

> "Based on the information presented in this Office Memo, as obtained from the reports in the Project Geology files, it is my belief that Emergency Spillway at Oroville Dam is a safe and stable structure founded on bedrock that will not erode."

All of the detailed information that was attached to the DWR memorandum was for the location of the emergency spillway crest structure, not for the hillside downstream. However, a review of the attached boring logs from the spillway crest location would lead to a conclusion that the rock contained areas of relatively deep weathering prior to excavation for the structure (See Appendix C). FERC received the 2005 DWR memorandum, and, to the IFT's knowledge, did not identify the inconsistency between the memorandum's conclusion and the attached borehole data.

In 2009, a study was undertaken by DWR to quantify the volume of material expected to erode upon use of the emergency spillway, specifically the soil and strongly weathered rock presumed to overlie non-erodible bedrock. The report resulting from this study [26] includes the following statement, citing the 2005 memorandum:

> "That memo (DWR 2005) also describes from an engineering geology point of view that there is 'an insignificant amount of top soil' overlying the bedrock. In that context, 'insignificant' means that even if the reservoir water were to flow over the Emergency Spillway, the water would very quickly encounter solid bedrock on the downstream side of the spillway, and as the thin mantle of soil would be eroded away, the rock would remain very resistant to erosion."

The study was then based on a thickness of erodible material on the hillside of 3 to 4 feet. However, the 2009 report includes an accurate summary of borehole information in a table documenting depths of "soil and decomposed rock" and "strongly and/or moderately weathered rock," up to 36.5 feet, which is in direct contradiction to the statement quoted above and the assumption of 3 to 4 feet of erodible material.

It appears that the interpretation of the right abutment geology from the 2005 and 2009 DWR memoranda, as described above, was presented to and generally accepted by the 2014 PFMA team (see Appendix F3 for discussion of the 2014 PFMA). In regard to Candidate PFM F2 – "PMF Event Occurring at Oroville Dam and Emergency Spillway is Overtopping and Head-cutting Occurs Initiating at the Feather River," the 2014 PFMA report [4] includes the following positive factor:

> "The rock between the Feather River and the emergency spillway is very competent and resistant to erosion."

With the geology being a major factor, the conclusion regarding this candidate PFM was:

> "The failure of Oroville Dam through head-cutting was not considered credible because of distance from the emergency spillway to the Feather River (approximately ¾ mile), the competence of the rock, and the lack of erosion that has been observed at the end of the FCO spillway channel."

In regard to candidate PFM F6 – "Scour of Soil and Debris During Flow over the Emergency Spillway Blocks the Feather River," the following positive factor is listed:

> "The slope below the emergency spillway has relatively little vegetation and surficial cover and the underlying bedrock is not subject to significant erosion."

This again reflects the established misinterpretation of the geology on the right abutment. In large part because of this established misinterpretation, significant erosion as a result of emergency spillway discharge was neither contemplated nor studied.

### 5.2.2 Incident Management Decisions

Following the February 7 service spillway chute slab failure and closure of the spillway gates, lake levels rose due to ongoing inflows, and it was readily apparent that the spillway gates would need to be re-opened, or the lake level would rise and overflow the emergency spillway weir. Either option presented risks. Refer to Appendix L for a detailed discussion regarding the decisions that led to the use of the emergency spillway.

DWR was managing the incident with consideration of at least the following four issues:

- Continued erosion at the service spillway progressing toward and compromising the service spillway headgate structure
- Tailwater from spillway discharges, combined with debris blockage in the river, potentially flooding the Hyatt Powerplant
- Continued erosion at the service spillway causing failure of a power transmission tower located to the right of the service spillway chute
- Potentially overtopping the emergency spillway crest structure for the first time ever in the project's history, with unknown consequences.

Should either the powerplant be flooded, or the transmission tower be lost, the powerplant would be out of service, and would not be able to release water downstream. This would result in significant long-term water management issues, mainly in view of environmental effects and water deliveries. It would also affect the logistics in water management during repairs to the damaged service spillway. However, the unknown risks of using the emergency spillway were also a major concern, in view of the major unexpected erosion of the service spillway foundation.

There were two divergent views being presented to the decision-makers:

- One favoring use of the emergency spillway as the best means to reduce the likelihood of the powerplant going off-line; this position was generally held by operations personnel and the DWR executives.
- The other favoring the use of the service spillway; this position was generally held by geologists, dam safety engineers and Incident Command personnel onsite. These individuals recommended increasing service spillway flows to whatever maximum safe flow could be allowed on the basis of observations of ongoing erosion.

From numerous interviews and review of incident notes, the IFT understands that DWR executives and managers were being told that flooding of the powerplant or loss of the transmission lines would put the powerplant out of service for at least months. Although true in the case of powerhouse flooding, it was subsequently found that the transmission tower could be replaced much more quickly than originally thought (see Appendix K). However at the time of the incident that was not known.

The IFT recognizes the difficulty of the very high-pressure situation and the many unknowns with which those on site had to deal during the incident. The IFT does not intend to criticize the hour-by-hour decisions that had to be made given these conditions, and is not at all critical of the decision to issue the evacuation order when erosion was observed progressing quickly toward the emergency spillway crest structure. Rather, it is the IFT's intention to consider how the decisions during the incident contributed to occurrence of discharge over the emergency spillway for the first time ever, and whether anything can be learned to improve such decision-making in similar situations in the future, recognizing that every emergency situation will be unique.

Decision-makers attempted to find the "sweet spot" in balancing the risks of operating the service and emergency spillways: i.e. releasing limited flows down the service spillway to both reduce ongoing service spillway erosion and prevent powerhouse flooding, while not overtopping the emergency spillway weir. The decisions were made on the basis of near continuous hydrological modeling of inflows to the lake, in view of upcoming storm patterns, and updated forecasting, which progressively indicated that greater service spillway flows would be required in order to not overtop the emergency spillway weir.

Hydrologic model uncertainty, although acknowledged, was intentionally ignored as the incident progressed. The use of mean model values was based on the belief that the risks of powerhouse flooding due to either the continued use of the service spillway or the use of the emergency spillway were essentially equivalent. Although perhaps true in regard to powerhouse flooding, the inherent additional risks associated with using the emergency spillway were not being fully considered. Specifically, the potentially unsatisfactory performance of the previously untested emergency spillway was not fully considered.

As seen in the Incident Command notes, there were two critical decision points that delayed releasing greater flows through the damaged service spillway, and essentially guaranteed that the emergency spillway would be overtopped, at least by a few inches:

- 1:16 pm on February 9 - the decision not to increase flows immediately from 35,000 to 50,000 cfs, but rather to "ramp up" more slowly. Flows more than 35,000 cfs were not allowed until about 6 hours later, and flows greater than 45,000 cfs were not reached until about 11 hours later.
- 7:05 pm call on February 10 - the decision to reduce flows from 65,000 to 55,000 cfs, and then holding to this decision until just before the evacuation order some 44.5 hours later.

These decisions likely resulted in a least a few feet of reservoir rise during the incident, based on approximate calculations by the IFT.

At these times, operations personnel were continuing to advocate to DWR managers their concerns regarding flooding of the powerplant and loss of the transmission tower due to further service spillway erosion, while DWR dam engineering and dam safety personnel were arguing strongly to avoid discharging over the emergency spillway, based on their review of geologic records from the files completed during the incident. The operational concerns carried the day.

In reviewing data provided by DWR (see Figure 5-1), the IFT found that, at the times these decisions were being made, there was no immediate threat of powerhouse flooding. In fact, when service spillway flows were reduced at about 7:00 pm on February 10, the tailwater level was actually falling and the threat to the powerplant was diminishing, even with the 65,000 cfs discharge. When the IFT asked a number of DWR personnel who were involved in the incident about the dropping tailwater level, they were surprised to hear this information. By their accounts, after the decision had been made to reduce discharge to 55,000 cfs, there was apparently little to no further discussion regarding whether service spillway discharges could be increased to reduce the risk of overtopping the emergency weir – this risk had been accepted. Further, the IFT learned that technical personnel had opined that the transmission tower was stable, and, due to better upstream rock quality, headcutting extending up to the service spillway headworks was no longer considered a major concern, based on their engineering judgment.

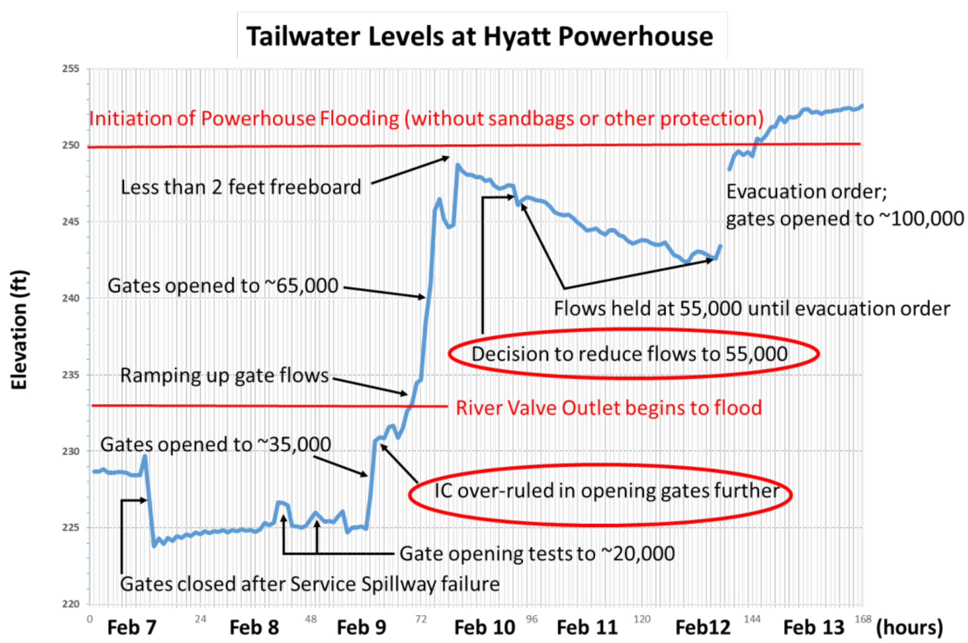


**Figure 5-1: Tailwater levels during the Oroville Dam spillway incident**

The incident is a story of "too little too late" in regard to releasing flows through the service spillway. Every delay in increasing flows meant that even greater flows were necessary later on to keep the lake level below the elevation of the emergency spillway weir. The IFT believes that all decisions were made with the best of intentions, but that the risk tradeoffs were not adequately assessed, due to an unequal balance between the understanding of the potential consequences and the uncertainties of the options under consideration.

# 6.0 GENERAL ORGANIZATIONAL, REGULATORY, AND INDUSTRY FACTORS

As noted in Section 5 above, the question of "why the incident happened" involves both factors which are specific to Oroville Dam and its spillways, as well as factors which more generally involve organizational, regulatory, and industry aspects. The factors specific to Oroville Dam and its spillways are discussed in Section 5 above, and the "general factors" are discussed below in this Section 6. As discussed in Appendix J, which describes the human factors framework and methodology used for this investigation, the findings and conclusions presented in this Section 6 are based on information obtained by the IFT from various sources, but predominantly from interviews and surveys.

The general factors discussed below are multifaceted and involve aspects such as confidence about the integrity of civil infrastructure, a reactive versus proactive approach to management of civil infrastructure, attention paid to spillways as compared to main dams, cost control pressures, priority for water delivery and power generation versus dam safety, technical expertise related to dam engineering and safety, information management for dams, human resources policy constraints related to staffing and continuing education, reliance on regulators and regulatory processes, limitations of regulators, and relationships within DWR.

## 6.1 Overconfidence and Complacency Regarding Infrastructure Integrity

The IFT found that DWR has been significantly overconfident and complacent about the integrity of its State Water Project (SWP) civil infrastructure, including its dams. This is not atypical among large dam owners. Contributing factors to this overconfidence and complacency are described below in this Section 6.1.

### 6.1.1 Quality of Project Design and Construction

A contributing factor to DWR's overconfidence and complacency was a somewhat widespread belief within DWR that the SWP was designed by the "best of the best" – a belief passed on through two generations to the current generation, and possibly increasingly mythologized by each generation (see Appendix K1). While it is true that DWR recruited nationally to hire qualified engineers and geologists from other organizations, it is unlikely that DWR was able to fill all of its key engineering and geology positions with the "best" people, given the rapidity with which DWR needed to scale up its organization during the 1960s.

The most relevant possible illustration of this aspect is that, as reported to the IFT in an interview, the principal designer for the Oroville spillways 1) was hired directly from a university post-graduate program, with prior engineering employment experience limited to one or two summers for a consulting firm, 2) had no prior professional experience designing spillways, but had received instruction on spillway design in university coursework on hydraulic structures, and 3) likely did not consult technical references regarding spillway chute design, and instead relied on notes from his university coursework in hydraulic structures. If this information is accurate, the IFT finds it striking that such an inexperienced engineer was given the responsibility of designing the spillways of what is still the tallest dam in the US.

### 6.1.2 Attention to Spillways Versus Dams

The IFT's general observation is that spillways have commonly been viewed as a "little brother" to dams in the dam engineering and safety industry. This was the case not just during the 1960s, but continuing until the February 2017 incident, and has arguably resulted in insufficient attention and a degree of complacency in the industry with regard to spillways. Both FERC and DSOD interviewees affirmed this observation with respect to their own organizations and the dam owners they regulate. A contributing factor to this limited attention to spillways has been that, in comparison with dams, spillway incidents have rarely resulted in loss of life, and therefore spillway incidents have been under-reported, resulting in lack of appreciation in the industry of the risks associated with spillways.

### 6.1.3 History of Prior Failures and Incidents

Another factor contributing to DWR overconfidence and complacency regarding the integrity of its dams may have been a lack of prior major incidents with DWR dams. However, DWR was aware of dam incidents and failures which could have tempered tendencies towards overconfidence and complacency, though the IFT found no evidence that DWR was influenced by those incidents and failures.

For example, California has had its share of major dam incidents, the most prominent being the 1928 failure of St. Francis Dam, which caused more than 400 fatalities and triggered the creation of DSOD (see Appendix K2). In addition, an incident in California which presumably should have loomed large in the minds of the designers of Oroville Dam was the failure of Baldwin Hills Dam in 1963, which resulted in at least three fatalities and was investigated by DWR in 1964.

Outside of California, other major dam failures and incidents which would presumably have been known to the designers of Oroville Dam included Malpasset Dam in France in 1959 (over 400 fatalities), Vega De Terra Dam in Spain in 1959 (over 100 fatalities), Vajont Dam in Italy in 1963 (over 2,000 fatalities), and Swift Dam in Montana in 1964 (over 25 fatalities). The era of design and construction of Oroville Dam was certainly one in which the potential for major dam failures was well known to dam engineers worldwide.

DWR itself experienced less consequential incidents at some of its dams, such as uplifting of the spillway chute slab at Castaic Dam. This was due to "heaving" of the foundation soil, rather than mechanisms similar to what occurred during the February 2017 Oroville spillway incident. In addition, interviewees noted that, in recent years, DWR has experienced "emergencies" with its infrastructure other than dams, such as its canals, on the order of one or two incidents per year. Therefore, while DWR had no prior major incidents with its dams, it is arguably not the case that DWR's incident rate for its overall civil infrastructure was low.

## 6.2 Reactive Approach to Civil Infrastructure Management

The IFT found that DWR was largely in a mode to react to problems with its State Water Project (SWP) civil infrastructure as they arose in the short term, rather than taking preemptive actions to proactively prevent problems. As with overconfidence and complacency, this is not atypical for large dam owners. This reactive approach was likely influenced by numerous factors, including cost control pressures, as discussed below.

### 6.2.1 Cost Control by Pressure from SWC

Owners of large dams and other civil infrastructure typically face substantial budget pressures, which can make it difficult to justify proactively spending money to prevent problems which have not yet occurred, whereas spending to address problems is rarely denied on a reactive basis once problems actually occur.

In the case of DWR, funding for the SWP was different from most other dam owners because, contractually, the actual costs of operating and maintaining the SWP were passed on to a group of 29 California public water agencies known as the State Water Contractors (SWC). The SWC were obligated to pay those costs in full, and DWR had discretion regarding how to operate and maintain the SWP and incur costs as necessary.

However, in practice, the IFT found that DWR was likely subject to significant pressures by the SWC to control costs, and these pressures were exerted in various ways, such as openly during discussions in meetings. Given that SWP costs paid by the SWC were ultimately passed on to the California public and farmers, it would be understandable that the SWC would themselves feel pressured to control costs. The IFT did not investigate these aspects in detail.

### 6.2.2 Cost Control by Limiting Size of State Government

In addition to cost control pressures exerted by the SWC specifically on DWR, there were also general pressures from the public to control the size and associated cost of California state government, of which DWR is a part.

A primary mechanism through which this pressure was exerted on DWR was in controlling the number of staff positions in DWR's organizational chart. Rather than DWR having discretion to hire new staff as it determined necessary, the IFT was told that new positions could only be added to the organizational chart with approval of auditors in the governor's office, and requests to add such positions were routinely denied. This resulted in a situation in which DWR often had sufficient budgets on paper to meet its objectives, but was chronically unable to fully make use of those budgets due to not having enough staff. This, in turn, also resulted in an increase in the amount of services contracted out to external consultants, which offset some of the cost control intended by limiting the size of DWR, but also entailed DWR needing resources to manage those consultant services.

The staffing situation was further exacerbated by other state human resources policies, which are discussed in Section 6.7.

### 6.2.3 Cost Control Internally in DWR

Based on numerous interviews, the IFT formed the impression that, independent of cost control pressures exerted by the SWC and state government, many decision-makers in DWR, especially in the Division of Operations and Maintenance (O&M), themselves had a strong desire to control costs. This likely stemmed from a sense of responsibility to reduce expense to the public, especially given the economic issues faced in California during the two decades prior to the February 2017 incident. This sense of responsibility became part of the culture of O&M in its Sacramento headquarters office.

### 6.2.4 Emphasis on Operations, Water Delivery, and Power Production

The IFT found that there was a well-intentioned desire within DWR to focus on operations, to meet the water delivery needs of the SWC and the California public and farmers, and the power production needs of the SWP. Water delivery was a primary objective of the SWP and DWR, and power production was needed to reduce the energy costs associated with delivering water (the SWP is a net consumer of power). Both water delivery and power production required vigilant attention with a relatively short-term focus.

However, this short-term attention likely came to some extent at the expense of more long-term attention to proactively preventing infrastructure problems and managing the associated risks. DWR needed to balance these goals of “production” versus safety, and lacked leadership and authority at the executive level which focused on finding the right balance (see Appendix K1).

In this regard, to the credit of DWR, the IFT notes that, starting in 2015, DWR was in the process of developing and implementing a relatively sophisticated and proactive approach to managing the SWP through an Asset Management Program in the Division of Operations and Maintenance (O&M), and an explicit goal of the program is to balance short-term and long-term priorities. However, DWR’s Asset Management Program was still in a relatively early developmental stage as of the February 2017 incident, and therefore not yet at a point where it could have made a meaningful difference in preventing the incident.

## 6.3 Insufficient Priority on Dam Safety

From surveys and interviews, the IFT heard diverse and somewhat divergent opinions regarding whether DWR gave sufficient priority to dam safety. This divergence of opinions indicates that groups within DWR, particularly the Division of Engineering (DOE) and Division of Operations and Maintenance (O&M), were often not “on the same page” regarding setting priorities.

This stems from the separation of DWR’s main civil/geotechnical and electrical/mechanical engineering resources into DOE “service providers” versus O&M “owners and operators,” which set up DWR to treat the risks associated with water retention and passage quite differently from those associated with power production. There has been no one in a position of authority specifically tasked with ensuring that the “balance is right.” There has also been a reliance on DOE to have technical expertise in all aspects of engineering associated with dam safety, but no apparent motivation to provide and undertake the training necessary to attain and maintain the appropriate level of expertise. This “two cultures” aspect is discussed further in Section 6.8.

Considering the pull of these two cultures, after careful consideration of the evidence, the IFT found that DWR gave insufficient priority to dam safety, but not to an extent which is atypical among large dam owners. As discussed in Sections 6.1 and 6.2, the priority DWR gave to dam safety was influenced by confidence instilled by the lack of prior major incidents at DWR dams, and the need to balance dam safety with other competing priorities, particularly controlling costs and delivering water and power without interruption. In addition, DWR's confidence in its dams was understandably bolstered by the fact that numerous independent consultants and both regulators had evaluated the facilities on a regular basis during the half-century prior to the February 2017 incident – at considerable expense – and had generally deemed the facilities, including the Oroville spillways, to be safe (see Section 6.4 and Appendices F2 and F3).

Historically, although DWR established a dam safety regulatory body in 1929, which was regarded as "the leading" state dam safety program in the United States in a 2016 peer review [27] (see Appendix K2), it took DWR more than another 70 years as a dam owner to establish an internal dam safety program, which the IFT found to be still in development as of the February 2017 incident (see Appendix K1). This indicates a general "hands-off" approach to dam safety at the Director and Deputy Director executive level within DWR, with lack of strong overarching "top-down" ownership of the dam safety program in DWR.

Instead, management of the dam safety program, which was organized as a Dam Safety Branch (DSB) in O&M, was left almost solely to the Chief Dam Safety Engineer (CDSE), although the CDSE was not in a position to best influence either investment or emergency decisions related to dam safety. There was no identified DWR executive that was specifically charged with overall responsibility for dam safety, and substantive regular communications regarding dam safety issues with upper management were lacking (see Appendix K1). In the opinion of the IFT, in a mature dam safety program, the CDSE should be reporting directly to whichever executive has overall dam safety responsibility, so that this executive is fully aware of dam safety concerns, including during emergency situations.

Although development of DWR's dam safety program was very late to start, the IFT believes that the program was maturing rapidly and on the right path (see Appendix K1). It was only very recently that appropriate levels of spending were being achieved and, as such, the "backlog" of dam safety issues requiring investigation likely appeared almost overwhelming to those at the working level in the DSB. In interviews, FERC personnel affirmed that "DWR has a lot to prioritize."

The IFT believes that the culture within DWR, and the relative immaturity of the dam safety program, also had a significant influence on past investigations into spillway condition and performance, as well as the outcome of the critical decision-making during the February 2017 incident (see Appendix L).

From a regulatory standpoint, DSOD focused on physical facilities and did not generally evaluate owner's dam safety programs. However, FERC did clearly recognize the importance of Owner's Dam Safety Programs (ODSPs) (see Appendix K1) and notes on its website [28] that the ODSP is:

> "the most important factor in maintaining safe dams and preventing dam failures. Dams with owners who do not have an effective ODSP represent a higher risk. The owner's dam safety program has been cited as a contributing factor in many dam failures including: the 1976 failure of Teton Dam, the 1986 failure of Upriver Dam, the 2003 failure of Silver Lake Dam, and the 2005 failure of Taum Sauk Dam."

In 2007, FERC provided guidance for dam owners to self-assess their ODSP, and FERC also indicated that owners should have their ODSP periodically independently audited, typically at a five-year interval. DWR did not develop an ODSP until 2013. A draft update was prepared in 2017, and DWR was in the process of initiating an audit in early 2017, which was deferred due to the February 2017 incident. The IFT found that the ODSP developed by DWR was much less detailed than would be expected from an organization dealing with major dams, and lacked specifics regarding overall responsibility and authority for dam safety in DWR, as well as details of accountabilities to ensure that required tasks were delegated and undertaken (see Appendix K1). Additional regulatory aspects are discussed below in Sections 6.4 and 6.6.

## 6.4 Dam Owner Reliance on Regulators and Regulatory Processes

The IFT found that DWR generally relied too much on regulators and the regulatory process to identify problems with its aging dam infrastructure, rather than proactively doing in-depth evaluations initiated by DWR and assuming full responsibility for the management and safety of its dam projects, as is required on both an ethical and legal basis. As with other factors, this reliance on regulators and the regulatory process was not atypical for large dam owners (see Appendices K1 and K2).

The various inspections and reviews which were performed to meet regulatory requirements were generally useful processes involving considerable effort and cost. However, they had limitations, and for Oroville Dam, they evidently did not adequately identify and evaluate the spillway failure modes which actually initiated during the February 2017 incident. In general, some key limitations of these inspections and reviews were as follows:

- **Regular inspections:** Inspections have not always accessed all portions of facilities close-up, and even when such access has been obtained, inspections typically have not been guided by checklists developed by knowledgeable specialists to help ensure that key failure modes were detected and properly evaluated (see Section 5.1.2). In addition, in practice, inspections have been primarily based on visual observation, which does not generally enable detection of "hidden" defects and deficiencies, such as problematic chute slab details and voids under slabs. Visual observations can result in recommendations for more advanced investigation, and testing methods, but application of these methods would require additional resources and funding.
- **Five-year inspections/reviews:** More substantial inspections and reviews conducted at five-year intervals, as mandated by federal and state regulations, have tended to focus on what has changed since the prior review, based on observations from inspection, surveillance, monitoring, and operations during the prior five-year interval, rather than more comprehensively reviewing long-term performance history and comparing the original design and construction with current best practices for design and construction. In fact, the IFT found evidence of simple "cut and paste" descriptions and claims from report to report. However, these reviews have often been relied upon by dam owners to serve as comprehensive reviews, despite not always having sufficient scope and funding for that purpose. The IFT believes that both the California state and FERC dam safety regulations have been somewhat ambiguous regarding how comprehensive the five-year reviews were

intended to be, which likely contributed to these reviews being overly relied upon but not sufficiently funded to serve as comprehensive reviews (see Sections 5.1.2 and 7, and Appendices F2 and K2).

- **Potential failure mode analyses (PFMAs):** PFMAs have been a useful supplement to five-year reviews, since PFMAs focus specifically on identifying and evaluating potential failure modes by using a diverse team. Nationwide, the initial FERC-mandated PFMAs conducted for many dams, including Oroville Dam, were found to not be sufficiently thorough, but in recent years new PFMAs for many dams, including the 2014 PFMA for Oroville Dam, were much more thorough. This type of explicit group brainstorming to search for "what can go wrong" at a dam can be a very helpful tool for safety risk management.

  However, in practice, PFMAs have tended to be limited by emphasizing uncontrolled reservoir release, and therefore have tended to "rule out" potential failure modes associated with component failures which would not necessarily result in uncontrolled reservoir release, but could still involve serious consequences for the dam owner and the public, as occurred during the February 2017 incident. In addition, similar to five-year reviews, the PFMA process has not always provided comprehensive reviews of facilities, yet dam owners have often relied on PFMAs to serve that purpose (see Sections 5.1.2 and 7, and Appendix F3).

While DWR and dam owners in general may have relied too much on regulators and regulatory processes, in the opinion of the IFT, DWR did have some justification in relying upon the numerous independent consultants who inspected and evaluated Oroville Dam and it spillways over the course of a half-century, with an expectation that they would have identified the types of major deficiencies which could lead to high-consequence incidents such as the February 2017 incident. The fact that the independent consultants fell short of doing this can partly be explained by limitations in the regulatory processes, as described above, and partly by factors which have resulted in some areas of insufficient technical expertise in the industry overall, as discussed in Section 6.7.

## 6.5 <u>Inadequate Information Management for Dams</u>

Management of information related to dams is an essential function for large dam owners, in order to properly perform inspection, surveillance, monitoring, and evaluation of dams and their appurtenances, and thereby help ensure safety, manage risks, and meet regulatory requirements.

The IFT heard varying opinions regarding the quality of DWR's information management for its dams. Combining those opinions with the IFT's own experience in obtaining information for its forensic investigation, the IFT found that DWR's information management needs improvement, and may have therefore contributed, over time, to the February 2017 incident, due to the right information not being in the right hands at the right times. This information includes some of the geology reports for the spillways, chute slab details and design calculations, service spillway construction records and design changes made during construction, history of service spillway chute repairs and their performance, and history of service spillway drain flows. In this regard, the authors of the 2014 PFMA report [4] for Oroville dam noted that: “Documents and data pertinent to the design, construction, maintenance, operations, and performance of the project facilities are distributed across several DWR Divisions and Offices. Recent efforts by DWR to consolidate and organize these documents to a central electronic repository or database should continue.”

From a regulatory standpoint, the IFT did not identify any specific requirement from DSOD regarding information management of dam owners. However, in support of its Part 12 process, FERC does have specific and detailed requirements for information management. One of those requirements is development of a Supporting Technical Information Document (STID), which is intended to be a “living document,” updated at an interval of not more than five years, which captures all of the key information needed to properly evaluate a dam and its appurtenances.

The IFT notes that the STID for Oroville Dam which was updated in 2014 did not include much of the information which would have helped identify the risks at the two spillways, and was therefore incomplete. Drawings with service spillway chute slab details, as well as numerical flood routings and water surface profile calaculations, were among the information which was not included in this STID. Drawings with slab details were included in the 2014 Ninth Part 12D report, however the IFT found no evidence that those details were reviewed.

As a model of information management for dams, the IFT believes that the system currently used by Reclamation as part of its Safety Evaluation of Existing Dams (SEED) process is a good example. Reclamation first produced loose-leaf bound “Data Books” that could have hard copies of reports inserted as they were completed. However, this system had limitations, and Reclamation converted to an all-electronic filing system where older documents that were scanned and converted to PDF were added to the electronic folder for a particular dam, as well as all new studies and reports that are converted directly to PDF. Within this filing system, reports can be cross-referenced in a number of ways (such as “Dam,” “Spillway,” “Geology,” “Inspection,” etc.) to produce a list of documents related to a specific subject. This system, when coupled with Recalamation’s electronic drawings, instrumentation data, and other electronic file systems allows reviewers access to a comprehensive set of files for each dam ranging from planning stages through post-construction studies, analyses, and modifications, from a personal computer. The system also makes available the status of all dam safety and O&M recommendations, and supporting documentation for each dam, which are also tracked electronically. Having such a system allows review teams access to files on any subject they choose to investigate further, and does not limit them to the conclusions from past reports, since they can research the source documents.

## 6.6 Limitations of Regulators

Beyond the limitations of the various inspections and reviews mandated by regulators (see Section 6.4), the IFT notes that regulators themselves face some of the same challenges as dam owners.

In the particular case of FERC and DSOD, both regulators had heavy workloads relative to their available staff, and due to being government agencies, they faced major bureaucratic constraints related to attracting, retaining, and adding qualified staff, as well as redirecting or terminating low-performing staff (see Section 6.7).

In addition, a further challenge faced by regulators is that, while they may, in theory, be able to reference "tough" regulations when dealing with dam owners for the purpose of managing risks to the public, regulators often have difficulty in gaining compliance from dam owners in practice, because there are limitations to the sanctions they can reasonably impose. The reality of the situation is that regulators must strive to be fair and reasonable in weighing a) the costs they impose on dam owners in order to perform studies and construction versus b) risks imposed on the public. Regulators must strive for this balance in a context which often involves considerable uncertainties. As a result, regulators are in a position which is not dissimilar to dam owners with respect to risk management decision-making.

In the specific case of DWR and Oroville Dam, interviewees from both FERC and DSOD indicated to the IFT that they sometimes had difficulty in getting DWR to comply with their regulatory expectations. However, interviewee opinions on this varied significantly and, on average, the regulators ranked DWR as approximately "mid pack" as compared to other large dam owners.

A related question which came up in interviews was whether the California state government's placement of DSOD within DWR impacted DSOD's effectiveness in regulating DWR as a dam owner. The IFT found that this organizational structure may have somewhat reduced the extent to which DWR took DSOD seriously as a regulator, as compared to FERC. However, it does not appear that DSOD treated DWR more leniently than other dam owners, and may have even gone out of its way to not be lenient, due to concerns that it could be perceived as being too lenient (see Appendices K1 and K2). One interviewee also noted that DSOD's placement within DWR may have given DSOD added "clout" when dealing with California dam owners other than DWR.

## 6.7 Insufficient Technical Expertise in Dam Engineering and Safety

The IFT found that DWR, and to some extent its regulators and independent consultants also, lacked necessary technical expertise in some areas of dam engineering and safety, with expertise generally tending to be stronger with dams than their appurtenant structures. This is not atypical for large dam owners nor for the industry in general, due to the era of major dam design and construction in the US being two generations in the past, and the resulting industry-wide shortage of highly experienced engineers. Owners such as DWR have also tended to not understand the complexity and multidisciplinary technical aspects of high velocity water conveyance structures, and instead sometimes incorrectly assumed that general civil and structural engineers without specialized training and experience can address these aspects.

A further issue specific to the dam engineering and safety industry is that the available technical literature is not organized as a set of unified and integrated national guidelines which are regularly updated. Instead, the technical literature in the United States related to dams, while very extensive, is scattered across several federal agencies, most of the states, and professional organizations such as ASDSO and USSD. Engineers assigned to address the safety of dams and their appurtenances must not only be familiar with these dam safety organizations, but must also keep up with the current state of practice in dam engineering. This can be a significant task for an engineer who is only occasionally assigned to work on dams and spillways.

This has created a situation in which only individuals who have spent considerable time searching and reviewing the technical literature (typically over the course of many years of career experience) and working with well-informed colleagues will be well acquainted with the literature, whereas others with less breadth and depth of dam engineering and safety experience and exposure will be at risk of not being aware of key information relevant to their work. The IFT found that this has generally been the situation with respect to industry knowledge related to design and failure modes of chute spillways in particular, with a resulting lack of spillway experts at sizable organizations including DWR, DSOD, and FERC. In the opinion of the IFT, it is likely that no individual who could truly be called a spillway expert – with corresponding specialized experience in hydraulic structures – had significant involvement in design, construction, inspection, evaluation, maintenance, or repair of the spillways of Oroville Dam prior to the February 2017 incident.

In addition to these industry-wide factors, the IFT found that the following were exacerbating factors specific to DWR's technical expertise in dam engineering and safety:

- **Organizational pride:** Understandably, DWR had a strong sense of pride in the organization and its historic achievements. However, this likely contributed to DWR overestimating its capabilities in recent decades with respect to dam engineering and safety – many DWR staff "didn't know what they didn't know," which created an issue of "unknown unknowns" (see Appendix K1).
- **Organizational insularity:** DWR is a somewhat insular organization, with a large percentage of its staff having spent most or all of their careers at DWR, and thereby not gaining much experience beyond the somewhat limited range of DWR's projects in recent decades. This issue applies to both engineers and geologists, and the IFT notes that geographic diversity of experience is especially important for engineering geologists; there is some truth to the adage that "the best geologist is the one who has seen the most rocks."
- **O&M emphasis on electrical/mechanical aspects:** DWR's internal policy has generally been that, starting a year after construction of a dam was complete, the Division of Operations and Maintenance (O&M) has functioned in the role of DWR's "dam owner" and key decision-maker for spending related to dam safety (see Appendix K1). However, O&M has generally had limited expertise in civil engineering aspects of dams, due to having a predominance of electrical and mechanical engineers, a focus on electrical systems and "moving parts," a limited number of civil engineers, and limited dam engineering experience of those civil engineers. In addition, O&M was impaired in drawing on the civil engineering expertise of the Division of Engineering (DOE), including its Dams and Canals section, because of its strained relationship with DOE (see Section 6.8).
- **DOE cost controls:** In order to control costs, DOE generally tried to keep its staff fully billable to active projects, rather than allocating sufficient budgets and overhead time for staff to develop and maintain expertise in various technical areas related to engineering and safety of dams and their appurtenances. The IFT does not believe that it would have been practical or realistic for all DOE staff to develop and maintain specialized expertise across

many technical areas, but DOE could create smaller groups to develop and maintain expertise in specific technical areas, serving as in-house consultants to apply that expertise to projects as needed.

Based on numerous interviews, the IFT believes that DOE's effort to control costs was partly motivated by a desire to be competitive on cost with external consultants, since O&M is the largest "client" in DWR for DOE engineering services, and O&M has the option of procuring these services from external consultants if it believes that DOE is not able to meet its needs (see Section 6.8).

- **Bureaucratic constraints in providing technical continuing education:** As a state agency, DWR faced bureaucratic constraints in providing technical continuing education and training to its staff, especially related to travel to attend conferences, committee meetings, training, etc. Such travel usually required planning long in advance, which was not always realistic, as well as approval from senior managers, which was often denied. However, DWR also did not adequately utilize less costly options for continuing education and training, such as webinars, technical publications, bringing training to DWR's offices, and networking with colleagues via phone, teleconferencing, web conferencing, email, etc. This resulted in DWR not effectively drawing on the technical expertise of organizations with substantial breadth and depth of dam engineering and safety experience, including federal agencies such as Reclamation and USACE. See Appendix K1 for additional discussion on this topic.
- **Bureaucratic general constraints related to staffing:** As a state agency, DWR generally faced major bureaucratic constraints in recruiting, interviewing, hiring, promoting, disciplining, and terminating staff. For example, the IFT was told that DWR job interviews must be structured such that all interviewed applicants must be asked the same initial and follow-up questions, and the questions must be phrased such that applicant answers are "scorable" – these interviewing constraints are very restrictive as compared to the private sector. In addition, policies related to disciplining and terminating low-performing staff imposed large documentation and time burdens on managers, including a significant risk to managers and DWR of having to deal with litigation. These constraints resulted in a somewhat higher than average percentage of low-performing staff at DWR, including in DOE, as compared to other large dam owners and large organizations in general, and this contributed to the cost control pressures described above.
- **Bureaucratic constraints related to specialized technical positions:** As a state agency, DWR faced major bureaucratic constraints in creating and adding senior and specialized non-management technical positions. Instead, nearly all available senior positions emphasized management responsibilities, and DWR's internal standardized tests used for determining eligibility for senior positions emphasized knowledge of human resources policies and management practices, rather than technical expertise. Related to this, DWR's main classifications for engineering positions were generic classifications of "Engineer, Water Resources," "Senior Engineer, Water Resources," "Supervising Engineer, Water Resources," and "Principal Engineer, Water Resources," with very limited provision for

more specific and specialized classifications such as “Dam Engineer,” “Hydraulic Structures Engineer,” “Hydraulic Engineer,” and “Dam Safety Engineer.” In addition, the Senior Engineer and higher positions were salaried positions, with no eligibility for overtime compensation, whereas the Engineer positions included compensation for overtime, which sometimes resulted in higher net compensation for individuals in the Engineer classification as compared to the Senior Engineer classification. These factors significantly diminished DWR’s ability to attract and retain highly-qualified technical specialists.

## 6.8 Strained Relationships Within DWR

Within large utility organizations such as DWR, it is not uncommon for there to be strain in relationships between design and construction groups, as well as between operations and maintenance groups, and DWR was not an exception to this. The strain between DWR’s Division of Engineering (DOE) and Division of Operations and Maintenance (O&M) was a recurring theme during the IFT’s interviews (see Appendix K1), has existed for decades, and was described in a 1996 DWR “organizational study” report. [29]

While this strain has been a significant problem in general, it is not the case that there has been a uniform degree of strain in the relationship between DOE and O&M. Rather, the extent of strain between DOE and O&M has varied considerably, depending on the specific groups in DOE and O&M which were working together and the specific projects on which they were working. While there have been cases where groups in DOE and O&M worked together poorly, there have also been cases where they worked together well.

The IFT believes that this strained relationship, over time, likely had a negative impact on DWR’s decision-making and deployment of technical expertise with respect to managing its civil infrastructure, including Oroville Dam and its spillways. It also clearly had a negative impact on the ability of DWR to meet the expectations of its two dam safety regulators, DSOD and FERC.

The IFT identified several key factors which contributed to the development and perpetuation of this strained relationship, including differences in the priorities and culture of DOE and O&M, historical shift and ambiguity of their respective roles, disparity of their relative influence in developing and managing infrastructure projects, and mutual dissatisfaction with their relationship of consultant and client. These factors are discussed below.

### 6.8.1 “Two cultures”

DOE and O&M have had fundamentally different priorities and cultures. DOE was focused on long-term reliability and safety of civil infrastructure and, as a matter of engineering ethics, was resistant to cost controls which could compromise that reliability and safety. By contrast, O&M had a shorter-term operational focus on delivering water and generating power on a cost-effective basis. As one interviewee described it, “one culture is engineering, another culture is O&M, which has some engineering in it” (see Section 6.2).

Due to these two different cultures – neither of which are inherently “wrong” – and the associated differences in areas of expertise, there was a communications gap between the two divisions, as well as a lack of mutual respect, with interviewees indicating that each side sometimes viewed the other as being “arrogant.”

### 6.8.2 Historical Shift and Ambiguity of Roles

Historically, there was a major shift in the sizes and roles of DOE and O&M.

When the State Water Project (SWP) was being designed and constructed, DOE (called the Division of Design & Construction prior to 1996) was very large and had a dominant role in DWR. Once construction of the SWP was tapering off, DOE began a process of necessary downsizing, and DWR’s focus turned to operating and maintaining the SWP, which resulted in substantial growth in the size and role of the O&M Division as DWR became increasingly “operations-centric.” As a result of this transfer of stewardship of the SWP, O&M functioned in the role of DWR’s “dam owner,” and DWR’s Dam Safety Branch (DSB) was eventually developed entirely within O&M, even though DWR also had a Dams & Canals section within DOE (see Appendix K1).

Some interviewees opined that, with this transition in roles, O&M took on too many responsibilities and DOE became too marginalized, which resulted in resentment on the part of some DOE staff. Other interviewees opined that the roles of each division were appropriate for DWR’s needs, and that DOE needed to adapt accordingly, rather than comparing with a past era.

Concurrent with this shift in roles of DOE and O&M during the past half-century, the IFT found that there was some ambiguity in defining the specific roles of each division, which contributed to the strain in their relationship. Recognizing this, DWR made efforts to clarify the roles and working protocols of the two divisions, as documented by several memoranda and a “Service Level Agreement” [30] which was prepared in 2014 and updated in 2016. However, in the opinion of the IFT, these efforts had limited effectiveness in resolving the fundamental and deeply entrenched issues which contributed to the strained relationship.

### 6.8.3 Development and Management of Projects Related to Dam Safety

With the historical shift in roles of DOE and O&M, there was also a pronounced shift in the influence of each division in developing and managing infrastructure projects, including projects related to dam safety.

With O&M functioning as DWR’s “dam owner” and containing its Dam Safety Branch (DSB), during the past decade especially, O&M also had a dominant influence in determining what projects related to dam safety should be initiated, what project budgets and schedules are reasonable, and who should provide the services on each project. DOE had some input to these processes and decisions, however O&M was effectively “in charge” of these decisions, and not always receptive to the input of DOE, sometimes partly due to O&M not fully appreciating civil engineering values and technical issues (see Appendix K1 and Section 6.6). This contributed to DOE’s feeling of often being marginalized and its technical expertise not being recognized, as described above in Sections 6.8.1 and 6.8.2. Cost considerations were also a factor in O&M’s assertion of a leadership role in managing projects, since O&M viewed itself as being accountable for costs and cost overruns on projects for which DOE provided engineering services (see Sections 6.2 and 6.8.4).

### 6.8.4 "Captive" Client and Consultant

DOE generally has not been providing engineering services outside of DWR, and O&M has been its largest client, representing close to half of DOE's total workload in recent years. To a significant extent, this put DOE in a position of being a "captive consultant" to O&M, and caused DOE to tend to view external consultants as unwelcome competition.

At the same time, DWR had established an understanding that O&M will generally give DOE the "right of first refusal" to provide engineering services for its projects before seeking external consultants to provide those services, and this arrangement was formalized by the "Service Level Agreement" described in Section 6.8.2. To a significant extent, this arrangement put O&M in the position of being a "captive client" to DOE.

Based on its extensive discussions on this topic with dozens of interviewees, the IFT believes that this mutually "captive" client/consultant relationship between O&M and DOE contributed to the strain in their relationship. Each side had developed grievances which, as noted above, were largely the same as described two decades ago in the 1996 DWR "organizational study" report [29]. Most of the IFT's interviewees in both O&M and DOE opined that the grievances by both divisions were largely valid, rather than placing "blame" only on one side.

Some of the key grievances from the O&M side were as follows:

- DOE tended to take O&M for granted as a client, and did not have a "customer" focus. O&M wanted to be treated as a valued and respected client by DOE, as it was treated by external consultants.
- Once DOE had been assigned a project, it tended to seek control of the work rather than partnering collaboratively with O&M.
- DOE tended to overestimate its technical expertise, and its expertise did not always compare favorably with some external consultants. This varied considerably among the various branches and sections of DOE, with some DOE groups being viewed much more favorably than others.
- Some of DOE's managers were viewed as not being strong technically, and/or not strong enough managers to elicit adequate performance from their staff, resulting in an excessive percentage of low-performing staff and associated cost increases (see Section 6.7).
- DOE's engineering designs did not always meet O&M's practical needs, and O&M felt that they were sometimes overdesigned, resulting in excessive construction cost. The State Water Contractors (SWC) also expressed this criticism of DOE's designs, sometimes directly to DOE and sometimes through O&M.
- DOE's costs for engineering services did not always compare favorably with external consultants. A contributing factor is that DOE tried to prevent its overhead from increasing by billing staff time to active projects, even if those staff were not working fully

productively on those projects. This sometimes resulted in assigning available staff to projects in order to keep them busy and billable, even if those staff had limited qualifications for those projects, which further drove up costs.

- DOE sometimes had difficulty meeting schedules.
- DOE was viewed as not sufficiently effective in managing the work of external consultants, possibly in part because it viewed those consultants as competitors.

From the DOE side, the grievances included the following:

- O&M tended to take DOE for granted as a consultant, and lacked understanding and consideration for the general challenges DOE faced as a consultant which is constrained by being part of a stage agency.
- O&M, and also the SWC, sometimes had unrealistic expectations of DOE, especially with regard to costs, because they lacked sufficient understanding of the technical aspects of civil engineering work and the associated provisions required to safely manage project risks.
- O&M often changed its project priorities, scopes, and schedules with limited notice, which made it difficult for DOE to plan workflow and project staffing. This, in turn, resulted in overhead and cost increases, and made it more difficult for DOE to meet schedules.
- Because O&M was generally in control of developing and assigning projects, DOE staff lacked opportunity to work on the same types of projects on a regular basis. This prevented development and maintenance of expertise with those types of projects.
- When O&M used external consultants for projects because DOE's cost was perceived as being too high, those consultants sometimes did substandard work which required revisions by DOE or resulted in an increase in construction costs or project risks. From the DOE perspective, external consultants lacked institutional knowledge related to the SWP, did not "take ownership" of their SWP work to the same extent as DOE, and therefore often did not develop designs and contract documents that sufficiently addressed the needs of DWR.

### 6.8.5 General Comments on the Strained Relationship

It is clear to the IFT that DOE and O&M have been engaged, for decades, in a difficult working relationship. Out of necessity for DWR and the SWP, and in turn the SWC and California public, the two divisions have needed to work together effectively, but have often had difficulty in doing so.

It would be inappropriate to point fingers and find fault with either division, or DWR overall, since this type of strain is somewhat typical of large utility organizations. However, it must be acknowledged that the strained relationship between DOE and O&M is a significant issue, and the IFT found that DWR's senior managers and executives did not appear to have a good grasp of the magnitude of the problem as it was experienced at the working level of staff and middle managers.

In the opinion of the IFT, serious and sustained involvement by DWR's senior managers and executives will be needed to make meaningful progress in improving this relationship, and experience shows that simply writing more study reports and memoranda is unlikely to provide much benefit. While it was not in the scope of the IFT's investigation to attempt to make specific recommendations regarding this issue, the IFT notes that substantial organizational changes may be needed to address the fundamental needs and grievances of both divisions, based on a realistic understanding of their respective cultures, goals, values, limitations, and circumstances.

# 7.0 LESSONS TO BE LEARNED

The IFT recognizes that, with the benefit of hindsight, it is much easier to determine "what went wrong" in terms of the physical sequence of events leading to the February 2017 incident, as well as the human judgments, decisions, actions, and inactions which contributed to that physical sequence of events. Therefore, the IFT has strived to avoid "hindsight bias" and a "blame" mindset, and has instead focused on understanding the contributing factors to the incident and the associated lessons to be learned.

This report section presents the IFT's findings regarding these lessons to be learned. The IFT has divided these lessons into two categories. First, industry-level lessons, which apply to dam safety practice in the United States are discussed in Section 7.1. Next, additional lessons, which apply more specifically to DWR are discussed in Section 7.2.

## 7.1 Industry-Level Lessons to be Learned for US Dam Safety Practice

The IFT offers six industry-level lessons to be learned that it has identified during the investigation. These lessons apply generally to dam safety practice in the United States and are related to:

- Physical inspections
- Comprehensive facility reviews
- Regulatory compliance
- Potential Failure Mode Analyses (PFMAs)
- Consideration of appurtenant structures
- Owners' dam safety programs and dam safety culture

The lessons identified by the IFT in these six areas are presented below.

### 7.1.1 Physical Inspections

In the IFT's opinion, physical inspections, while a necessary part of a dam safety program, are not sufficient to identify risks and manage safety. At Oroville Dam, more frequent physical inspections would not likely have uncovered the issues which led to the spillway incident. The warning signs of these issues were already known to DWR and others, but had been accepted as normal conditions.

In dam safety practice, physical inspections are typically visual inspections from accessible locations and do not directly provide insight into latent conditions which cannot be detected by visual inspection. For the Oroville Dam service spillway, the observed slab cracking and the drain flows had become accepted by DWR, DSOD, FERC, and external consultants as "normal" conditions, and the slab details which increased its vulnerability to failure went unnoticed. As long as the physical inspections revealed no detected change in the observed conditions, no concerns were identified. For the emergency spillway, it had become assumed that the hillside downstream of the crest control structure was comprised of non-erodible rock with 3 to 4 feet of soil cover. Here also, visual inspection alone would not have provided information to change that opinion.

Instead, comprehensive reviews of available information and knowledge of how spillway failure modes develop, as discussed below, are needed to understand the significance of the conditions observed at the service spillway, the vulnerable chute slab details, and the susceptibility to erosion at the emergency spillway.

When warranted by uncertainties and risks, results and observations of physical inspections, guided by knowledge gained from comprehensive reviews, should be supplemented by subsurface and non-destructive testing, such as video, geophysical methods, and ground penetrating radar, to investigate and gather data on actual conditions within or beneath components such as chute slabs. In the case of service spillways, such as at Oroville Dam, this could include focused investigation of characteristics and extent of concrete delamination, spalling, and cracking; condition of drainage systems; and condition of foundation areas where strongly weathered bedrock exists. Although subsurface investigations can be costly, they can provide essential information, and can be planned and budgeted in advance. The results will better inform owners and regulators and provide needed support for subsequent five-year reviews, PFMAs, and risk analysis processes.

The lesson regarding physical inspections is pertinent not only to the parties involved at Oroville Dam, but more broadly to United States dam safety practice. The overwhelming majority of dams in the United States are regulated by state dam safety organizations. Although a few states, such as Colorado and New Mexico, have begun to incorporate file reviews and failure modes analyses into their regular activities, most state regulators center their programs around physical inspections and what can be identified through those inspections. Incorporating periodic file reviews and failure modes evaluations into state dam safety programs will likely require both additional resources and reallocation of available resources. However, without such change, we resign ourselves to continued occurrence of incidents such as happened at Oroville Dam in February 2017, with a frequency and severity which will likely be deemed unacceptable to dam owners, regulators, and the public.

### 7.1.2 Comprehensive Facility Reviews

Periodic comprehensive reviews of original design and construction, performance, maintenance, and repairs are needed for all features of dam projects. These reviews should compare the various features of the project with the current state of practice to answer the following questions:

- Is the feature consistent with current design and construction practice?
- If there are variations from current practice, do they compromise the structure and present a risk of failure or unsatisfactory performance?
- If there is not enough information available to make those judgments, is the potential risk sufficient to justify further study or evaluation?

The reviews should be:

- Thorough, taking advantage of all available information.
- Critical and independent, rather than relying largely on the findings of past reviews.
- Completed by people with appropriate technical expertise, experience, and qualifications to cover all aspects of design, construction, maintenance, repair, and failure modes of the assets under consideration.

The IFT has not seen any indication that such a review was ever conducted for the service spillway chute at Oroville Dam since the original construction, and the ambiguity of state and federal regulations regarding the intended comprehensiveness of reviews likely contributed to this. Such a review would likely have “connected the dots” and informed the evaluation and PFMA processes, by identifying the physical factors that led to failure of the service spillway chute and the damage that occurred at the emergency spillway, namely:

- Subsurface geologic conditions that were not appropriately addressed in design and construction
- Design shortcomings with respect to to the current state of the practice that made the spillway chute susceptible to concrete cracking, spalling, and uplift
- Construction procedures, decisions, and changes to the designs that exacerbated the shortcomings of the design
- Drain flows during spillway discharge that were well beyond what were intended in design and beyond observed drain flows at other spillways
- Chute slab repairs that were generally superficial and possibly detrimental, rather than designed to reliably and durably withstand high velocity flows, thermal effects, and other loading conditions
- Geology, topography, infrastructure, and other conditions on the hillside downstream of the emergency spillway that made the hillside susceptible to substantial and rapid erosion

Had these conditions been understood, evaluations completed as part of the PFMA or other efforts would have likely identified the vulnerabilities of the spillway chute and emergency spillway, and, possibly, failure modes associated with these structures would not have been judged very unlikely and ruled out of further consideration. At the very least, it could have been recognized that more data and analyses were required to better understand the potential failure modes.

Although the specific issues during the February 2017 incident were related to the two spillways, this lesson should be applied more broadly to dams and appurtenant structures. The Oroville Dam incident involved the spillways – the next incident or failure could involve another type of appurtenant structure or component of a dam.

In addition, the specific lessons to be learned from the Oroville Dam incident should not be focused only on chute spillways. The IFT notes that DSOD and FERC have required owners to complete reviews of concrete chute spillways after the Oroville incident, and other regulators or owners may be requiring similar reviews. While it is appropriate to conduct such reviews, unlined emergency spillways should not be forgotten in the review process. Many dams have emergency spillways that include unlined sections comprised of rock or soil, and many of these spillways have never been used or have not yet experienced their design loads. It is the expectation that, when called upon, these emergency spillways will perform without experiencing erosion that could threaten release of the reservoir, and this expectation could be based on unfounded assumptions. The dam engineering community's understanding of erosion of soil and rock has improved significantly since many of these structures were designed and constructed, and reviews to assess the expected performance of these structures should be completed.

### 7.1.3 Regulatory Compliance

Compliance with regulatory requirements is not sufficient to manage dam owners' and public risk.

Current dam safety regulatory requirements are generally focused on preventing failures involving uncontrolled release of stored water. They can also be too focused on extreme events with low likelihoods of occurrence, rather than events that are more likely to occur over the life of the dam. As dramatically demonstrated at Oroville Dam, serious incidents can occur for more frequent events which do not necessarily lead to uncontrolled release, but still have significant impacts to the owner and public, such as a) limitations on an owner's ability to control the reservoir, b) costs of emergency management and repairs, c) damage to or loss of resources and project benefits, d) environmental damage, e) impacts on society, f) damage to reputation, and g) third-party liability.

Current PFMA and risk analysis processes are also focused on uncontrolled release of reservoir water, and generally do not include development of scenarios for non-release incidents that can result in the same impacts noted in the previous paragraph.

In general, it must be recognized that regulators have an essential role in management of dam safety, but do not have the resources nor the primary responsibility for managing dam safety. That responsibility, both legally and ethically, rests with dam owners.

### 7.1.4 Potential Failure Modes Analyses (PFMAs)

The PFMA process has been recognized as a very useful tool in dam safety management, helping many dam owners to better understand their dams and the associated risks. However, the Oroville Dam spillway incident has identified some shortcomings of the PFMA as it is currently being used in dam safety practice. Some specific suggestions for improvements to the PFMA process are presented in Appendix F3. Some more general considerations are discussed in the following paragraphs.

In practice today, PFMAs appear to be limited mainly to consideration of potential failure modes that lead to uncontrolled release of the reservoir. This can lead to potential failure modes with significant consequences short of reservoir release being ruled out of further consideration. In the case of Oroville Dam, the 2014 PFMA team essentially identified the two failure modes which initiated in February 2017, but ruled them out, in large part because they were judged to be unlikely to lead to release of stored reservoir water. For the service spillway, this judgment may have been correct. For the emergency spillway, it is not clear whether the conclusion was correct. However, irrespective of whether the judgments were correct, the initiation and partial development of the failure modes had significant impacts on the owner and the public. By ruling out these potential failure modes, they may have been removed from any further consideration in subsequent studies, including future PFMAs.

It is also important to recognize that failure of a component can result in compromising an owner's ability to manage the reservoir. For example, the service spillway chute failure slab at Oroville Dam compromised DWR's ability to operate the spillway as planned, ultimately leading to discharge over the emergency spillway and the subsequent evacuation of about 188,000 people.

The three PFMA efforts for Oroville Dam demonstrate that the results of a PFMA are dependent on the quality of the information and the knowledge and experience of the team members. The process relies on engineering judgment to subjectively determine the relevance of the various failure modes. When a failure mode is ruled out in large part because of a single factor, such as the interpretation of the geology, the assessment of the supporting information for that factor needs to be diligent. In the IFT's opinion, that was not the case for the assessment of spillway area geology during the 2014 PFMA or during geologic assessments completed by DWR in 2005 and 2009.

For potential failure modes related to the service spillway chute, it was important to understand the design and construction of the chute and to understand spillway chute failure modes. It appears that review of the spillway chute design and construction was not completed for any of the PFMA efforts, or at any other time, and the PFMA teams did not include spillway design or spillway failure mode specialists. For structures of the size and complexity of Oroville Dam, it may be appropriate to use different teams of specialists for different parts of the project, rather than a single team for the entire project.

In general, although a very useful tool, which is likely quite adequate for a majority of dams, the current PFMA process can have difficulties in properly characterizing risks for large or complex systems, including accounting for human and operational aspects in failures. By defining failure modes as a linear chain of events, there can be a tendency to oversimplify complex failure modes involving multiple interactions of system components. Knowledge of the full range of dam safety risks resulting from all operational aspects is required for an organization's managers to decide on appropriate actions to manage those risks.

In addition, the current PFMA process does not explicitly consider how broader organizational factors, such as culture and decision-making authority and practices, can contribute to failure. For such situations, it may be necessary to supplement the typical PFMA process with other approaches as used in other industries, which can possibly better address these complexities and operational aspects.

Overall, the IFT believes that a critical review of the current application of the PFMA process in dam safety is warranted. Such a review should compare the strengths and weaknesses of the PFMA process with risk assessment processes used in other industries and by other federal agencies. Evolution of "best practice" must continue by supplementing current practice with new approaches, as appropriate.

### 7.1.5 Consideration of Appurtenant Structures

It is the IFT's impression that the 770-foot high embankment dam and the service spillway headgate structure at Oroville Dam received much more attention during the PFMAs than did other components of the facility, such as the service spillway chute and the emergency spillway. However, the February 2017 incident demonstrates that the spillway structures are significant structures themselves. The spillway chute is a 500-foot high structure intended to perform adequately under high discharge volumes with high velocities. Although the common perception was that the emergency spillway would be needed only during "extreme" events, the operation plans indicated that the emergency spillway may operate more frequently (see Appendix F3). Moreover, as this and prior incidents demonstrate, severe erosion at unlined spillways can occur at relatively low discharges.

It is the IFT's opinion that appurtenant structures can sometimes be eclipsed in dam safety evaluations by the main dam structure, and it is important that appurtenant structures receive the attention appropriate to their importance and their associated risks.

### 7.1.6 Owner's Dam Safety Program and Dam Safety Culture

Dam owners must develop and maintain mature dam safety management programs which are based on a strong "top-down" dam safety culture.

Along with the regulatory requirement for a Chief Dam Safety Engineer, there should be one executive specifically charged with overall responsibility for dam safety, and this executive should be fully aware of dam safety concerns and prioritizations through direct and regular reporting from the CDSE, to ensure that "the balance is right" in terms of the corporation's investments.

## 7.2 Other Specific Lessons to be Learned for DWR

The IFT believes that all of the industry-level lessons identified above in Section 7.1 are applicable to DWR. In addition, the IFT also identified several lessons which are specific to DWR. These DWR-specific lessons are based primarily on the IFT's evaluation of information gathered during interviews with more than 75 people, including current and retired employees of DWR, DSOD, and FERC. The IFT found that these lessons can be categorized into four areas. Progressing from broader organizational aspects to considerations more specific to dam safety, these four areas are:

- Organizational culture and internal working relationships
- Appropriate staffing for technical positions
- Technical expertise related to dam engineering and safety
- Dam safety program and risk management

The suggested lessons in these four areas are discussed below. The IFT provides these lessons not to criticize DWR, but rather to offer suggestions which may be helpful to DWR.

### 7.2.1 Organizational Culture and Internal Working Relationships

The IFT believes that DWR has been somewhat overconfident and complacent about the integrity of its State Water Project (SWP) civil infrastructure, including its dams, and should, therefore, shift its organizational culture in a direction that reflects more humility and vigilance regarding the risks associated with this infrastructure. As demonstrated by the February 2017 incident, there are risks associated with this infrastructure mainly having been designed and built a half-century ago, and, therefore, potentially having design and construction features that may be judged inadequate based on current states of practice and knowledge. This infrastructure also has risks due to aging and associated potential failure modes that develop over the course of years and may not be readily detected using conventional inspection and evaluation methods.

Similarly, the IFT believes that DWR has been somewhat overconfident regarding its technical expertise related to dam engineering and safety. Rather than associating itself with the accomplishments of its engineers and geologists from two generations ago, DWR should instead shift its organizational culture in a direction of more humility regarding its expertise and an orientation towards being more of a “learning organization.” This is discussed further in Section 7.2.3 below.

Another broader organizational aspect which DWR needs to address is the strain in the relationships between some of its internal groups, especially between the Division of Operations & Maintenance (O&M) and Division of Engineering (DOE). These strains have been present for decades, and past efforts to alleviate them have not had significant and lasting impact. While these types of strains are not atypical in the industry, they do potentially impact dam safety, and, therefore, need to be actively addressed by DWR, with involvement of staff at all levels of the organization, including DWR’s executives and senior management. To some extent, “silos” will be unavoidable in a large, complex, and multi-objective organization such as DWR, and so DWR should learn to better communicate and coordinate effectively across silos.

### 7.2.2 Appropriate Staffing for Technical Positions

The IFT found that DWR has been faced with very significant bureaucratic constraints with respect to maintaining a size and composition of its technical staff that fits its evolving needs. These constraints have substantially inhibited recruiting and hiring of qualified individuals, promoting staff to senior technical positions, and redirecting or terminating chronically underperforming staff. Additional inhibiting factors have included lack of overtime compensation for senior staff, and use of generic position titles which do not reflect the specialized roles and expertise of technical staff. These constraints have significantly impaired DWR’s ability to develop and maintain organizational technical expertise, control costs, meet schedules, and maintain morale.

The IFT believes that executives and managers in DWR, including the Division of Safety of Dams (DSOD), should be provided with greatly increased autonomy, discretion, and flexibility with respect to defining position descriptions; adding, removing, merging, and modifying technical positions in its organizational charts; recruiting, interviewing, and hiring staff; promoting staff to senior technical positions; compensating staff for overtime and specialized qualifications; and redirecting or terminating chronically underperforming staff. Rather than the decisions of DWR’s managers related to these aspects being "micro-managed," the IFT believes that DWR should be provided with reasonable staffing budgets which should be allocated and used at the discretion of DWR's executives and managers.

### 7.2.3 Technical Expertise Related to Dam Engineering and Safety

Each dam owner should have access to a level of interdisciplinary breadth and depth of technical expertise that is sufficient to assure management of the risk profile associated with its dam portfolio. In the case of DWR, the risks associated with Oroville Dam and its other dams are obviously quite high, as evidenced by the large number of people evacuated during the February 2017 incident.

The IFT believes that, prior to this incident, DWR did not have sufficient breadth and depth of expertise to manage the risk associated with its dam portfolio, and should therefore increase its expertise related to dam engineering and safety. The following are suggested measures to help accomplish this:

- **Communication, Coordination, and Staffing:** As noted above, communication and coordination between DOE and O&M should be improved, including between the DOE Dams and Canals section and the O&M Dam Safety Branch. The Dams and Canals section should learn more about dam safety management, the Dam Safety Branch should draw more on the technical expertise of the Dams and Canals section, and the Dam Safety Branch should continue to develop the technical expertise of its own staff. In addition, as noted in Section 7.2.2 above, the general human resources constraints on DWR's staffing of technical positions should be substantially reduced.
- **Cultivating In-House Specialized Expertise:** DWR should cultivate development of teams of specialists in various aspects of dam engineering and safety, supporting them by allocating time and funding for them to learn about and keep up with evolving states of practice. These staff should be provided with compensation and position titles that are commensurate with their specialized expertise. It should be recognized that it is not reasonable or prudent to rely on generalist civil and structural engineers to make engineering judgments and decisions for dams and appurtenant structures which are large, complex, and/or high-risk facilities.
- **Interaction with the World Beyond DWR:** As an organization, DWR should interact more with the national and international dam engineering and safety communities, in order to learn from others and identify best practices. This interaction could include attending and presenting papers at conferences, participating in technical committees, reading and contributing to technical publications, and networking with colleagues, including counterparts who have similar roles at other dam owner organizations.
- **Enhanced Continuing Education and Training:** DWR should generally increase the level of the continuing education and training provided to its technical staff involved in dam engineering and safety. In addition to options which involve travel, DWR should also increase its use of less costly options such as participating in webinars, bringing training to DWR, review of technical literature, and networking with colleagues via phone, email, etc.

### 7.2.4 Dam Safety Program and Risk Management

Although the DWR dam safety program is still in development, the program is on the right path and has been maturing rapidly in recent years. This progress should continue. In that regard, it is important that the dam safety program, particularly the Dam Safety Branch (DSB), have adequate funding and also adequate, qualified staff. The IFT found that most of the senior staff of the DSB, and certainly the Chief Dam Safety Engineer, have been highly dedicated and have worked long hours without overtime compensation. While this dedication is laudable, this situation is neither sustainable nor in the interest of DWR's dam safety program. Instead, the IFT believes that the DSB should have sufficient staff and funding to identify and manage dam safety issues on a proactive basis, rather than merely struggling to keep up with regulatory requirements on a reactive basis.

From an organizational structure standpoint, the IFT discussed the placement of the DSB in DWR's organizational chart with numerous interviewees. The IFT heard diverse opinions regarding where the Dam Safety Branch should be positioned. This is clearly an issue which requires consideration of numerous factors and their tradeoffs, and the IFT suggests that DWR evaluate whether a change in the positioning of the DSB and CDSE is warranted. Regardless of whether a change is made, the IFT emphasizes that is it essential that DWR have clear "top-down" leadership on dam safety from a designated and accountable DWR executive, and that the Chief Dam Safety Engineer have a regular, direct line of communication with this particular executive.

The IFT also believes that DWR should continue with development of its Asset Management Program, with dam safety and risk-informed decision-making incorporated as an integral part of this program. The development of an appropriate prioritization scheme is central to this effort. This will facilitate proper resource allocation and risk management for DWR's dam portfolio, in the context of the overall State Water Project infrastructure and DWR's multiple organizational objectives. To support both this Asset Management Program and DWR's dam safety program, the IFT suggests that DWR continue to work towards improving its information management, and should aim to develop a state-of-the-practice information management system for its dams and other infrastructure.

DWR should also contemplate what could improve its approach to dam safety, over and above simple regulatory requirements. A review of dam safety program procedures and components utilized by others, both nationally and internationally, would be appropriate, and could include consideration of detailed governance, implementation, and Operations, Maintenance, and Surveillance (OMS) manuals.

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[30] *Service Level Agreement (SLA) Between the Division of Operations and Maintenance And Division of Engineering*, The Natural Resources Agency Department of Water Resources, established November 17, 2014 (V2.0 September 2016).

# Appendix A

# Design and Construction Information

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## 1.0 GENERAL DESCRIPTION OF THE SPILLWAYS

The spillways are located on the right abutment of the dam and have two components: a gated flood control outlet (FCO) structure (service spillway), and an uncontrolled emergency overflow spillway (Figure A-1). The ungated, concrete emergency spillway is a 1,730-foot long overflow weir structure located to the right of the flood control outlet.

The discharge capacity for the two spillways the original design maximum water surface, Elevation 917 is 296,000 cfs for the service spillway, and 350,000 cfs for the emergency spillway [A-1].

### 1.1 Service Spillway

The general description of the service spillway provided in this appendix was derived from the as-built drawings. The service spillway consists of an unlined approach channel, a gated headworks structure, and a lined concrete chute extending to just above the river channel and terminating with a short terminal structure.

The headworks structure has a total of eight top-seal radial gates, with 140.7 feet of effective crest length between piers. It is comprised of two concrete monoliths, each containing four radial gates and five piers. The gates are identical except for the trunnion support beams at each end of the monoliths (Bays 1, 4, 5, and 8). The trunnion beams at the ends of the monoliths were modified in 2002 to eliminate single-sided trunnion girder cantilever action. The individual gates are 17 feet, 8 inches wide and 33 feet, 6 inches high. The sill elevation of the service spillway headworks is at elevation 813.6 feet at the upstream end and elevation 811.80 feet at Station 12+97.24 where it meets the spillway chute. The tops of the gates (in the closed position) are at approximately elevation 847 feet.

The chute is 3,057.75 feet long and 178.67 feet wide, extending from the downstream edge of the headworks structure to the downstream end of the terminal structure about 100 feet above the Feather River, a drop of about 500 feet. The first 1,000 feet of the service spillway chute downstream of the headworks slopes at 5-2/3 percent grade, after which the chute transitions through a vertical curve to a slope of about 24.5 percent for the last 1,455 feet.

Four large, reinforced concrete chute blocks are located at the terminal structure in the last 55 feet at the downstream end of the chute, to disperse the spillway flow before it enters into the river. Spreading the flow over a larger area with the blocks will decease the erosion in the bed as compared to a concentrated jet from a chute without blocks.

The 178.67-foot wide service spillway includes 160 feet of chute slabs that were designed to be a minimum of 15 inches thick. There are four lanes of chute slab, numbered 2 through 5, from right to left. Each lane is 40 feet wide, and separated by longitudinal contraction joints. The two outside lanes are placed against the wall bases, which are also separated from the adjacent chute slabs by longitudinal contraction joints. Transverse chute slab contraction joints are spaced at 100 feet upstream from Sta. 33+00. It appears that the spacing of transverse contraction joints changed to 200 feet downstream from Sta. 33+00. Specifications drawings called for ¼-inch filler material to be placed in the top inch of each contraction joint. The specifications also required transverse joint spacing at 50-foot spacing. Intermediate (50-foot) joints were formed by sawcutting fresh concrete to help initiate cracking at these stations between the contraction joints. This detail changed the joints at the 50-foot stationing to control joints (joints with continuous rebar). Waterstops were not provided at any of the joint locations, except the transverse joint between the chute and the headworks structure at Sta. 12+97.24, and at the longitudinal joints between the chute blocks in the terminal structure.

Transverse herringbone drains were placed beneath the chute slab at 20-foot spacing in the upper flatter portion of the chute, and 25-foot spacing in the steeper lower chute. These drains were placed protruding into the concrete slab, rather than in a trench fully beneath the slab. The final configuration of these 6-inch diameter drains encased in a gravel envelop left 7 inches of concrete cover above the drains. The specified slab thickness was a minimum of 15 inches.

### 1.2 Emergency Spillway

The emergency spillway is also located on the right abutment, to the right of the service spillway. The emergency spillway consists of two sections: a 930-foot long gravity ogee weir on the left side and an 800-foot long broad crested weir on the right side that is downstream of a parking lot. The crests of both sections are at elevation 901 feet, which is 1-foot above the maximum normal operating reservoir elevation of 900 feet. The maximum height of the emergency spillway crest structure is approximately 50 feet in the ogee weir section. Water flowing over the emergency spillway crest structure then passes over natural terrain to the Feather River. The emergency spillway was activated for the first time during the 2017 flood event and spillway incident.

## 2.0 SPILLWAY DESIGN

This section discusses the design of the gated service spillway (FCO) and emergency overflow spillway. Figure A-1 is a plan view of these two spillways.

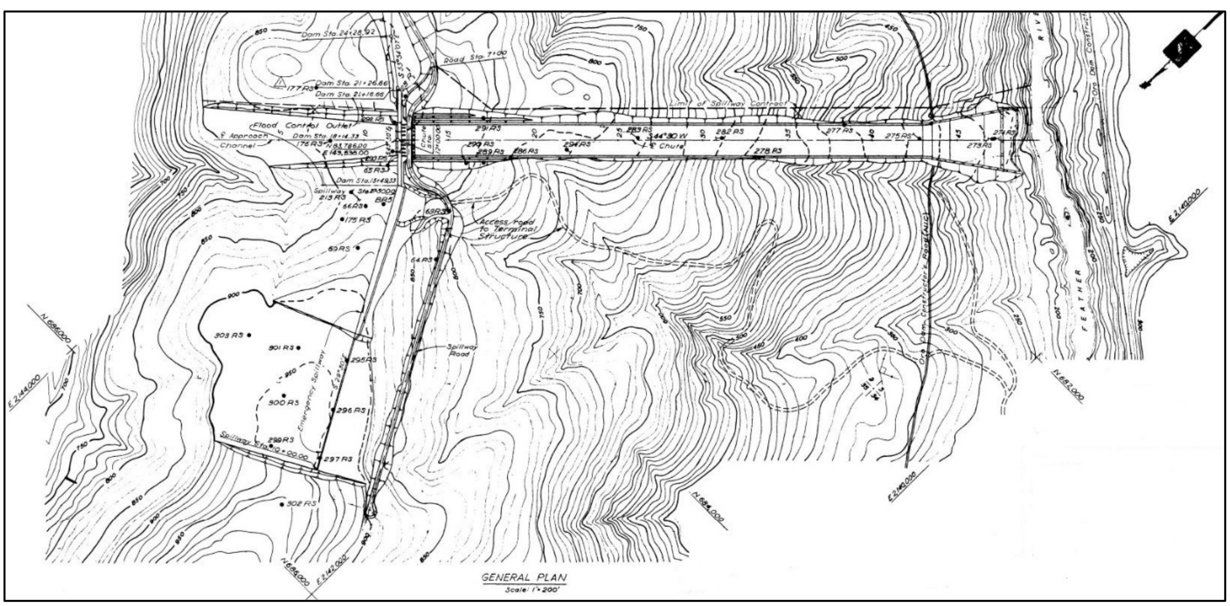


**Figure A-1: Pan View of the Gated Spillway (top) and Emergency Spillway (bottom)**

## 2.1 Service Spillway Design Details

The following discussion addresses several aspects of the service spillway design, namely foundation drains, the chute slab, slab joints, and foundation anchors. Details can be found on the drawings from Specifications No. 65-09 [A-2]. Most of the details are shown on drawing A-3B5-3, A-3B9-1, A-3B9-2, and A-3B9-4.

The discussion in this Spillway Design portion of this Appendix includes information related to the "original design" as portrayed in the bid Specifications No. 65-09 and the "as-built" changes portrayed in revised drawings that were prepared following construction. While the Independent Forensic Team (IFT) has attempted to determine when and why changes were made, documentation of these decisions was not always available, despite an extensive record search by Department of Water Resources (DWR) staff. The IFT interviewed an individual associated with the original design. Although some design details on the original bid specifications drawings and as-built drawings did not seem to match this individual's recollection of those details, earlier working versions of the drawings could not be located for review by the IFT. It is assumed that changes were made to the drawings throughout the review process, and that some of the details on the final bid specifications drawings differed from those initially proposed by the design engineer. The individual associated with the original design also indicated that DWR was compartmentalized during the design and construction, and that designers did not talk to geologists, and once the designs were turned over to construction, the designers were not involved in any design changes that took place during construction. The revision blocks on the drawings seems to support this claim because the original designers signatures do not match the initials for the drawing revisions.

As can be seen in the discussion under the Construction of the Service Spillway section of this Appendix, documentation for some changes to requirements in the specifications paragraph has not been located, and the IFT is only aware of these changes through construction reports and photographs.

### 2.1.1 Foundation Drains

The service spillway chute design included an underdrain system to relieve uplift pressures acting on the chute slab, as well as to drain the backfill adjacent to the chute walls. The spillway drainage system consists of two elements:

- An underdrain system to collect seepage and control uplift pressures within the foundation of the service spillway gate structure. A portion of this system is connected to the chute underdrain, while a separate portion of this system discharges water onto the chute.
- An underdrain system to collect groundwater seepage and control uplift pressures beneath the chute floor slab and in the backfill adjacent to the chute walls.

The bid specifications chute slab underdrain system consisted of 4-inch perforated vitrified clay pipe (VCP) lateral drains that were to be connected to 6-inch unperforated drains passing under the chute wall base, and then connected to 6-inch unperforated collector pipes, located outside the chute at the base of the walls (Figure A-2). The 4-inch lateral drains were to be placed in a gravel envelope, that was then to be covered with polyethylene sheeting to prevent concrete contamination during placement. The backfill outside the chute walls was to be drained by 8-inch perforated drains that connected to 6-inch unperforated collector pipes that discharged through the chute walls back into the chute (Figure A-3).

The collector drains were modified during construction, based on concerns from the Consulting Board [A-3]. During an April 1966 site inspection, the Consulting Board raised concerns regarding the potential for blockage of the longitudinal collector drains that could lead to a build-up of pressure under the slabs. As a result, the pipe size was increased, all pipes were given a gradient for positive drainage, and vertical risers were provided to permit back flushing. On August 1, 1966, the drawings were revised.

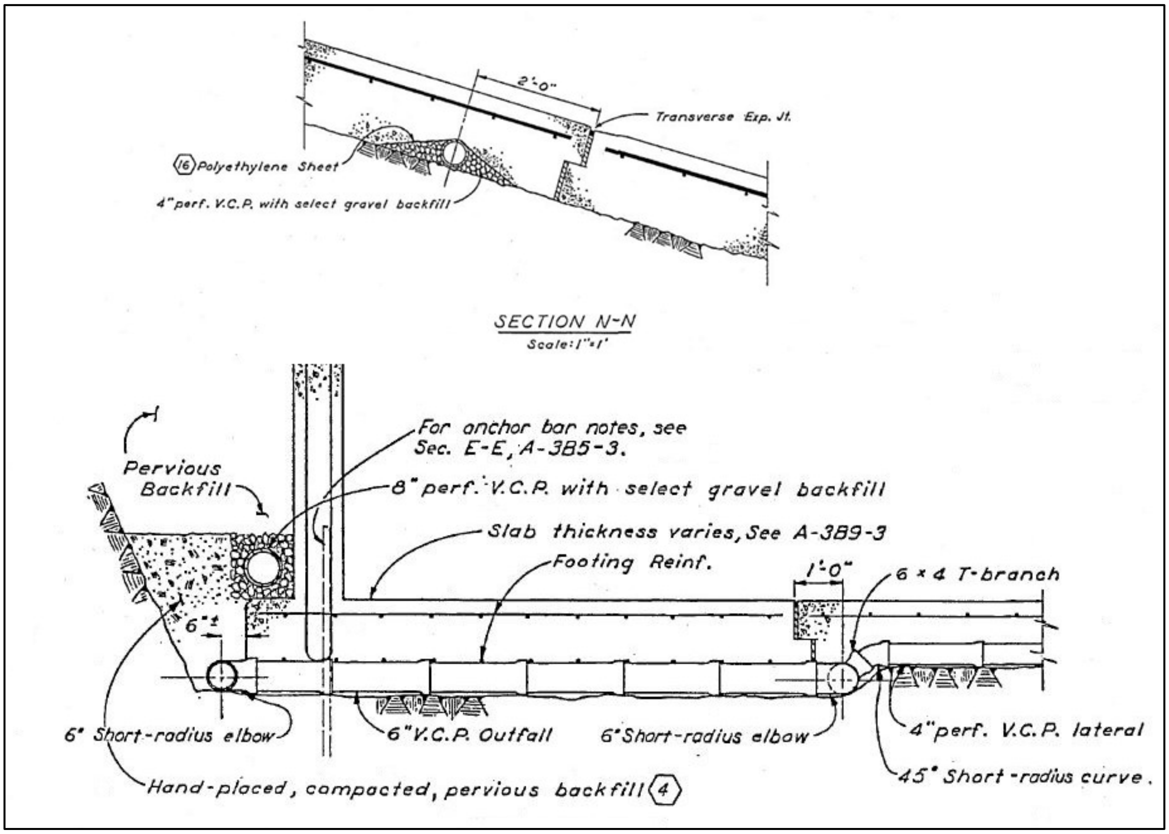


**Figure A-2: Original Lateral Drain Details from the Bid Specifications Drawing A-3B9-1**

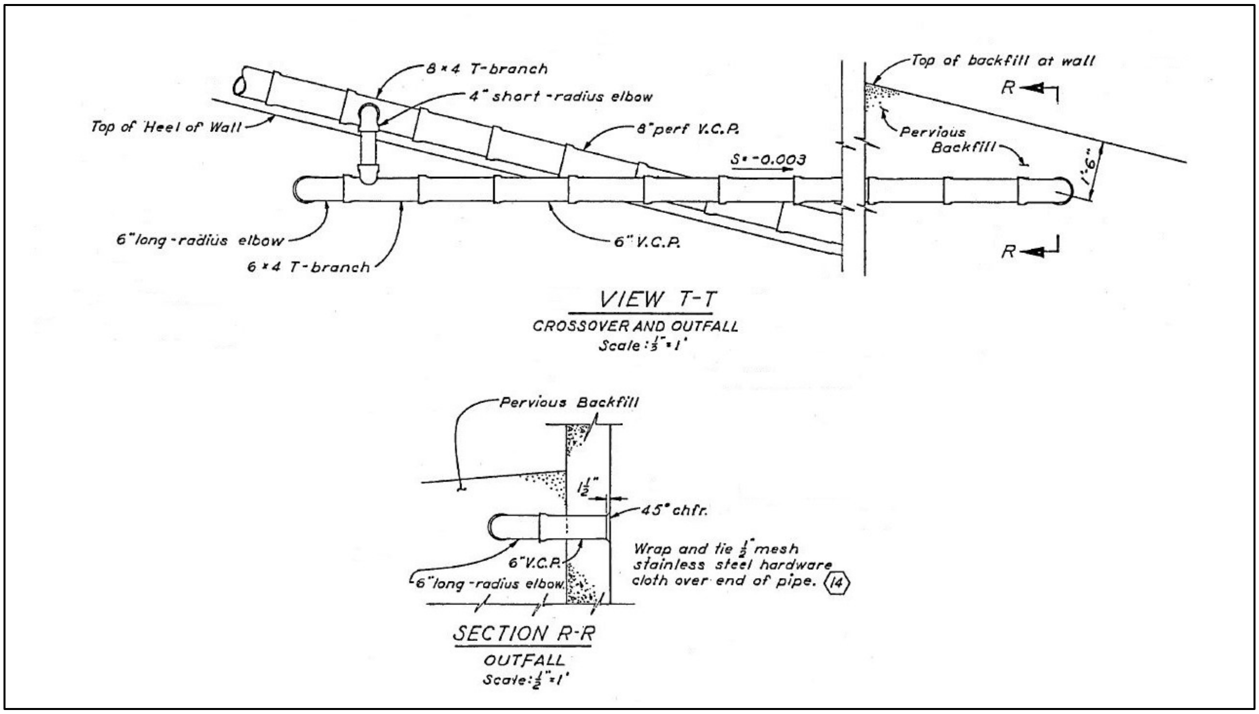


**Figure A-3: Original Collector and Backfill Drainage Details from the Bid Specifications Drawing A-3B9-1**

Based on a Consulting Board recommendation, the lateral drains were increased to 6-inch diameter drains, spaced at about 20 or 25 feet (depending on location) along the chute slab length, and embedded in a gravel envelope at the foundation surface, directly under the chute floor slab, and the collector drains were increased to 12-inch diameter (Figure A-4). The lateral drains were also to be slopes by 1% to 4% for drainage. This detail resulted in the drains being arranged in a "herringbone" pattern in plan (Figure A-5). This change was discussed in the Final Construction Report [A-4], and shown on the as-built drawings.

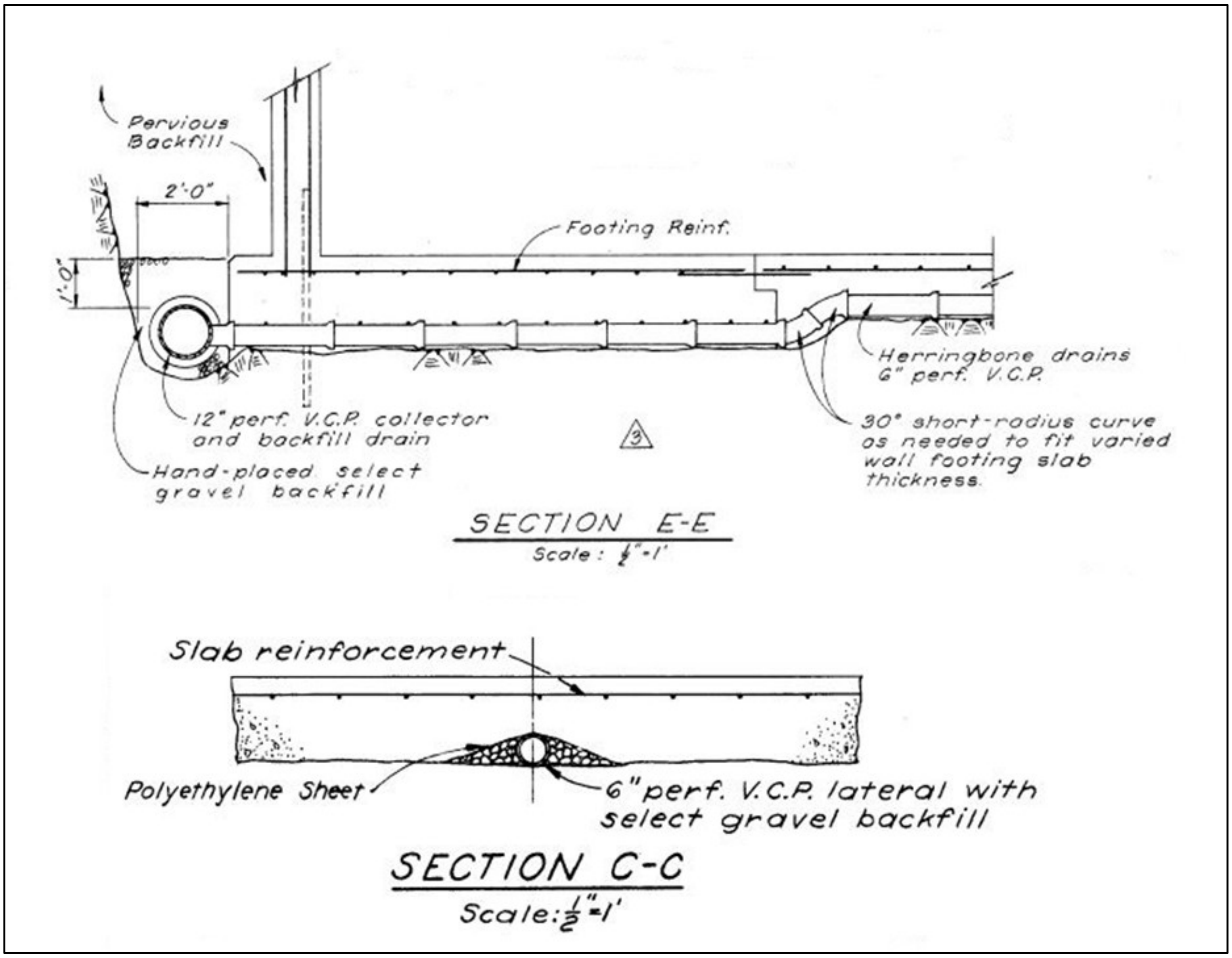


**Figure A-4: Upper Spillway Chute Drains Showing Herringbone and Collector Drainage (top) and Herringbone Installation Beneath the Chute Slab (bottom) from As-built Drawing A-3B5-3**

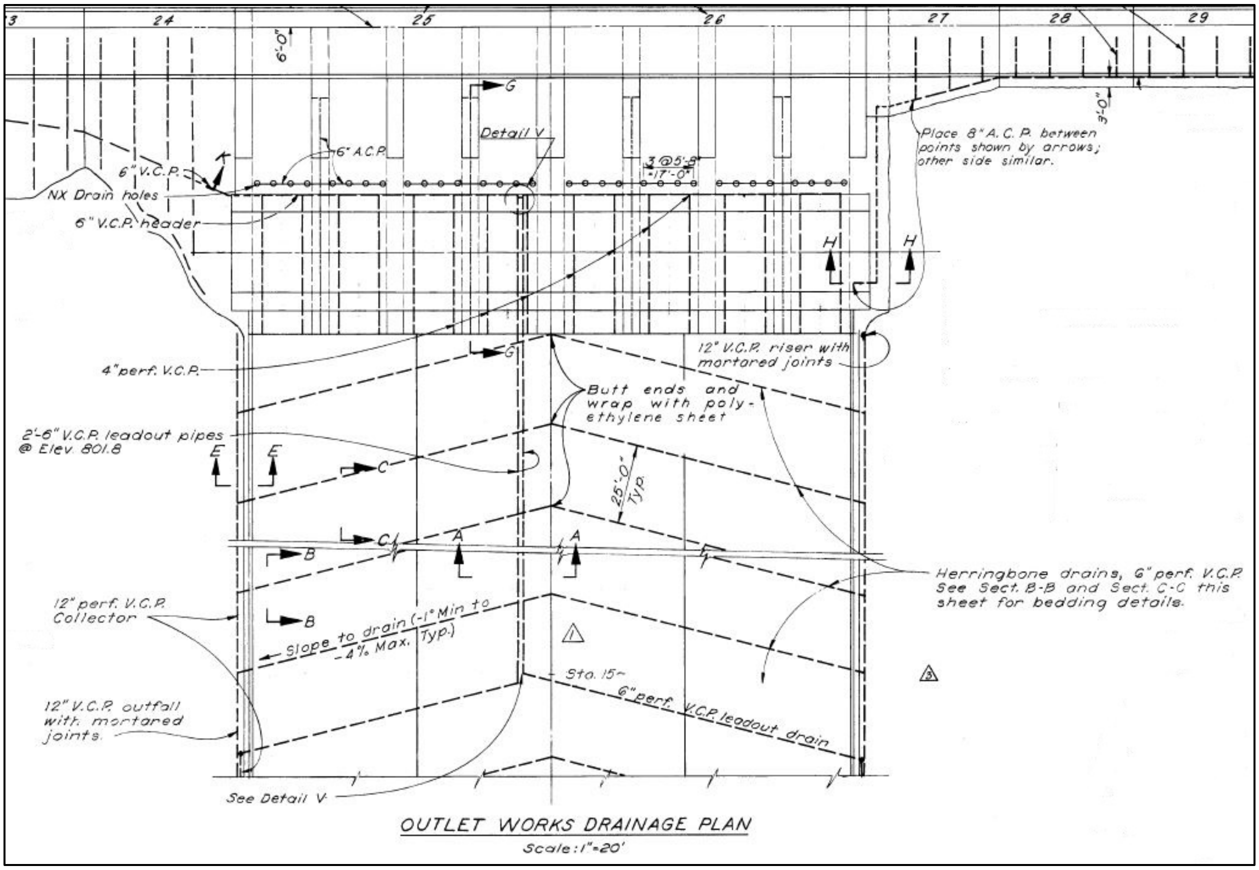


**Figure A-5: Plan View of Headworks (top) and Upper Spillway Chute Showing Herringbone and Collector Drainage (from Drawing A-3B5-3)**

With the as-built changes, the collector pipes begin as 12-inch perforated pipes, replacing the separate 8-inch perforated backfill drains, then extend downstream as non-perforated VCPs, set at a flatter angle than the chute slope, and eventually discharge into the chute through outfalls that penetrate the walls (Figure A-6). Rather than having a single, common collector pipe on each side of the chute, there are many independent collector pipes on each side of the chute, each draining a section of the chute and wall. In each the upper and lower chute (upstream from Station 27+00), there are six separate pairs (one on each side in a pair) of collector pipes. The underdrain system can collect water from different sources including:

- Groundwater seepage through the spillway chute foundation;
- Leaks through the concrete slab; and
- Water infiltration into the wall backfill from surface runoff, rain, or seepage.

The original design of the herringbone drains was intended to collect groundwater seepage from the foundation. The Final Geology Report [A-5] indicates that flows from foundation springs before construction were no greater than 2 to 6 gallons per minute (gpm). It was expected that flows could increase as the reservoir was filled, but the drains could remove much larger flows than the foundation features were expected to produce. There is no known documentation to indicate that the designers intended to collect leakage through open joints and cracks in the chute floor slab and side walls. Although offsets at lateral contraction joints (discussed below) were intended to prevent chute flow from entering the joints, high flows related to stagnation pressures at offset joints (Appendix B) were not well understood at the time of the Oroville service spillway design. As-built drawings (Figure A-6), as well as the underdrain schematic drawing prepared by HDR [A-6] (Figure A-7), show details of the drainage system.

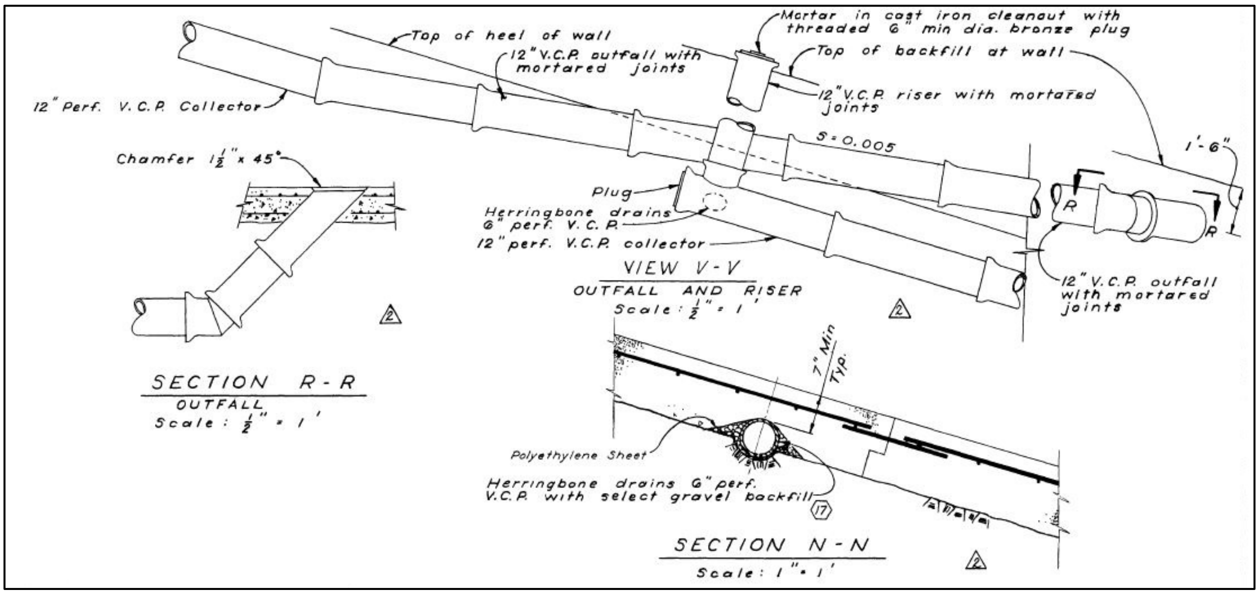


**Figure A-6: Herringbone (bottom) and Collector Drain Details (from Drawing A-3B9-1)**

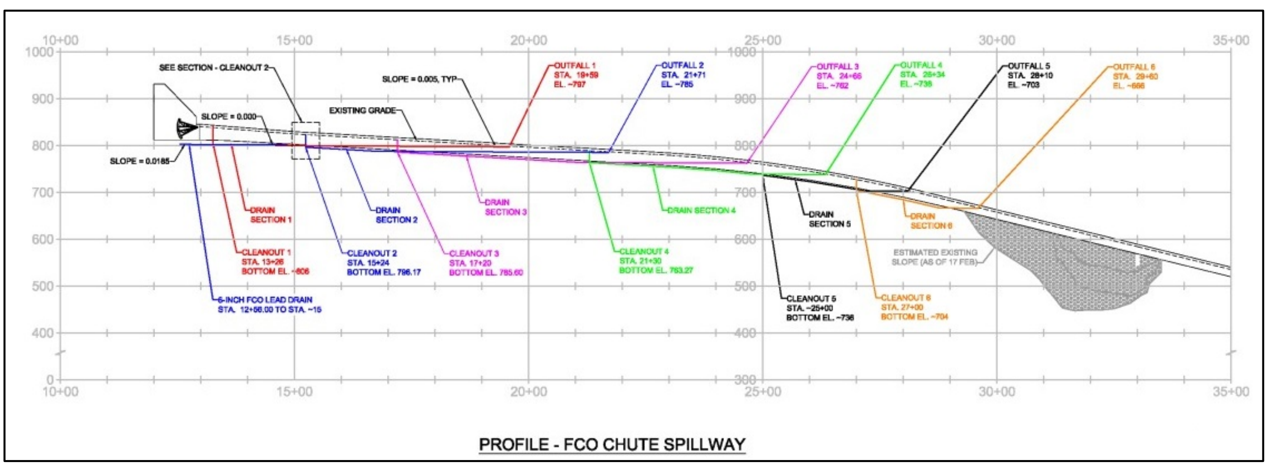


**Figure A-7: Underdrain Schematic Drawing prepared by HDR [A-6]**

Following the as-built design changes, the perforated lateral drains were to be placed within the bottom half of the spillway chute slab. After the drain size was increased to 6 inches, the minimum concrete cover over the drains was reduced to 7 inches above the pipe (Figure A-6). The perforated pipe was to be laid within an envelope of select gravel that was to be mounded over the top of the pipe. Perforations were to be oriented to face the foundation surface. Polyethylene sheeting was to be placed over the pipe and gravel envelope. Where foundation overexcavation was required, gravel contained by wooden formwork was to be placed between the foundation and the pipe (Figure A-8). Pipe laid within the limits of the concrete was to have caulked joints. The drains were to be field adjusted to have a 1 to 4 percent slope for drainage, as discussed above. However, from cracking patterns, it appears that the drains were placed uniformly and parallel to each other in a herringbone pattern. Construction photographs show that in areas of deep overexcavations, the drains were connected to the foundation through gravel columns that were constructed by placing gravel in sonotubes, see Figure A-9 (Photo 40 of the Final Geologic Report [A-5]), or in wooden formwork.

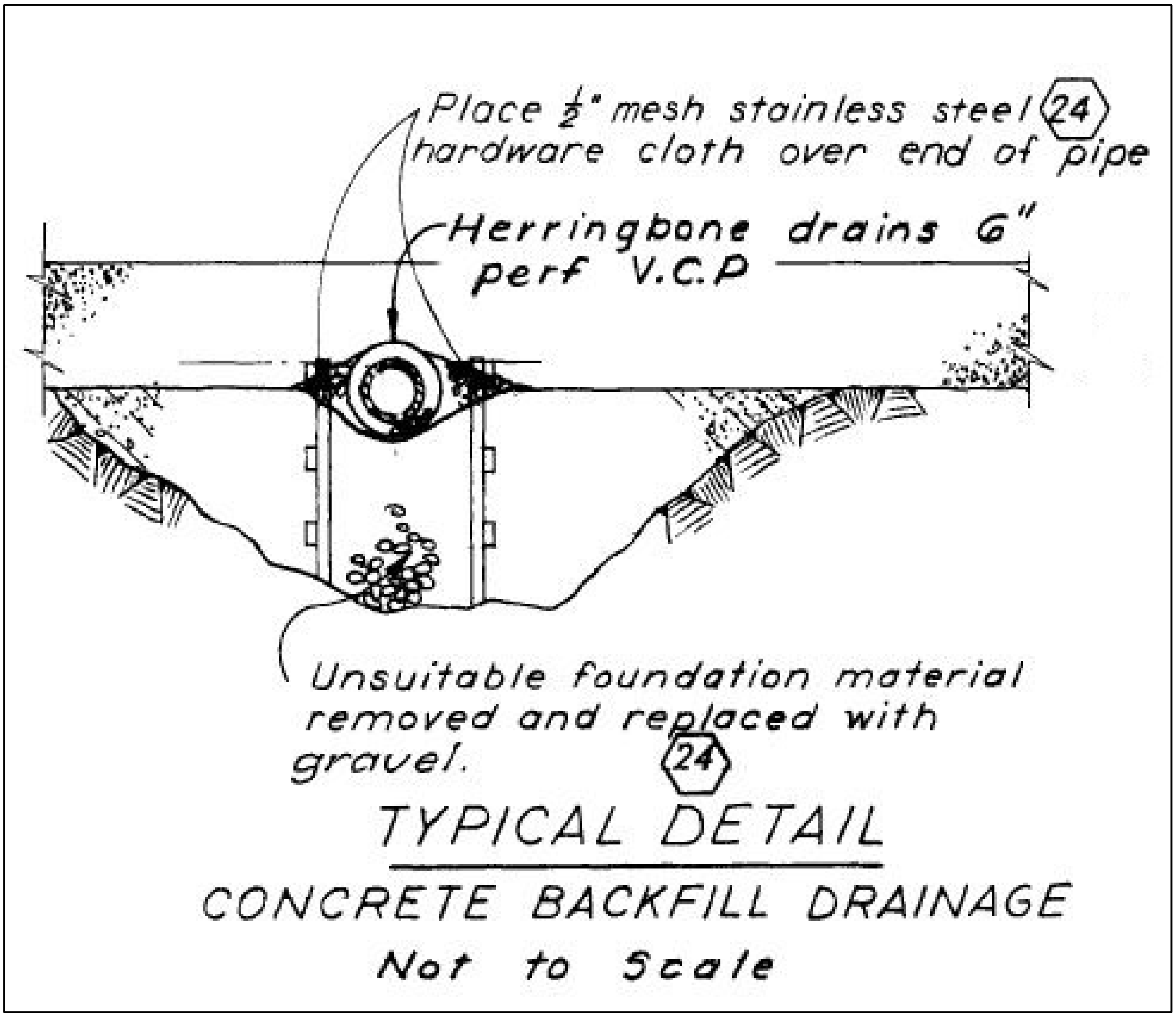


**Figure A-8: Herringbone Treatment Where Overexcavation Occurs (from Drawing A-3B9-1)**

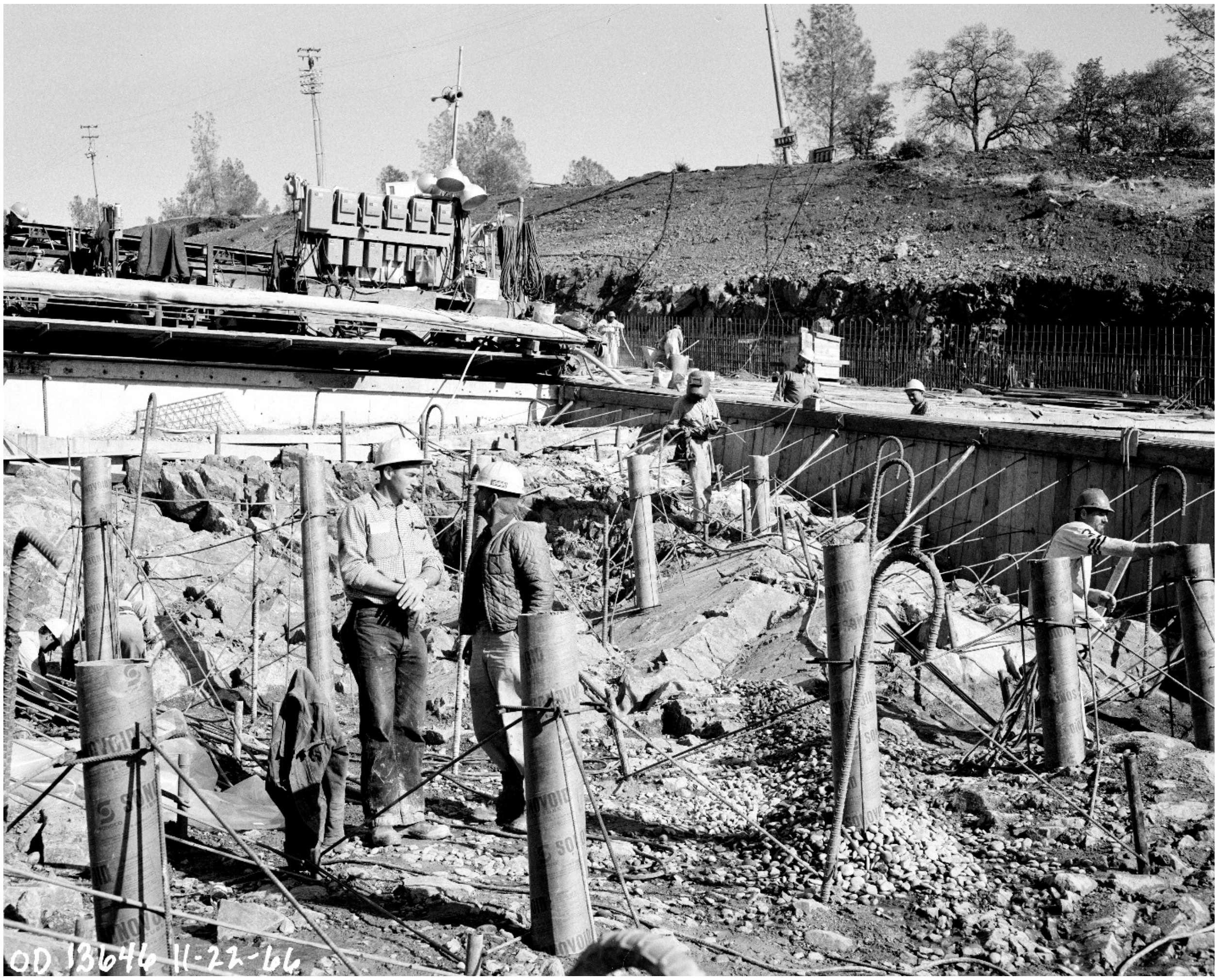


**Figure A-9: Gravel-filled Sonotubes Being Installed to Connect Herringbone Drains to the Foundation through the Backfill Concrete**

Spillway drawings were sent to the California Division of Dam Safety (DSOD) and reviewed by them in February 1967 [A-7]. Review comments indicated a concern that surface water (outside the spillway chute) would add to the flow carried by the collector drains, and could result in lifting of the floor slabs as pressure backs up into the herringbone drains. It was recommended that impervious material be placed over the pervious backfill surrounding the drains to prevent surface water from entering the drains. This modification apparently was not made as there were no subsequent revisions to the drawings.

As-built drawing details show the 6-inch diameter herringbone drains connecting to 12-inch perforated VCP collector drains that were originally separate from the collector drains. The slopes of the perforated collector drains apparently match the slopes of the chute wall bases along the lengths of wall where the herringbone drains connect to the collectors (Figure A-6). Based on the May 6, 2017 Underdrain Inspection Report by HDR [A-6], individual collector drains collect herringbone flows for varying lengths of the chute floor, as indicated by the cleanout pipe stationing, which corresponds with the upstream ends of the collector pipe sections. At the end of each collection zone, the 12-inch perforated pipe connects to an unperforated pipe, and the slope is flattened so that the pipe can exit through the chute wall downstream. Drawings show the centerline of this exit point to be 18 inches below the top of the wall, but field observations indicate that this distance may be greater than 18 inches.

There is almost no redundancy in the Oroville service spillway drainage system. Only one collector drain was provided to service many herringbone drains, although the herringbone drains for each side of the chute are connected at the chute centerline. If the collector drain is plugged, there is no alternate drain to relieve pressure in the entire zone of herringbone drains serviced by the one collector drain. This detail does not seem consistent with other wide spillway drainage systems being designed at the time Oroville Dam was designed, as discussed in Appendix E, that have several intermediate longitudinal drains, or a drainage gallery connected to the transverse drains. The herringbone drains are also very long, at half the spillway width or about 90 feet, without any interconnection. If a herringbone drain is plugged, water in the foundation would need to work its way to another herringbone drain 20 to 25 feet away, by flowing through channels at the foundation contact or within the foundation. Surprisingly, neither the Consulting Board nor the DSOD reviews seemed to address this deficiency due to lack of collector drain redundancy in the design. However, it is believed that they expected that only a small amount of groundwater would enter the herringbone drains.

The herringbone drain design places the drains within the 15-inch concrete slab, such that there is only 7 inches of concrete cover above the drains (not including the bell at the connections), see Figure A-6. While this creates a problem in terms of concrete cracking above the drains, it also places the drains well above the foundation should overexcavation occur. Since the drains were intended to relieve pressure from groundwater in the foundation, it would seem that a better design would have been to place the drains on the foundation or in a foundation trench, as was being done at other dams at the time Oroville Dam was being designed (see Appendix E). There apparently was no consideration that the drains placed within the concrete slab would lead to concrete cracking and additional drainage requirements. Also, with polyethylene sheeting being placed over the top of the drain to prevent concrete contamination of the drains and gravel envelope, flow into the cracks occurring above the drains would be diverted away from the drain immediately below. Apparently, the designers, design reviewers, and staff making as-built changes to the drawings did not understand the concern related to water from the flow surface passing through cracks and joints into the foundation that other spillway designers at the time seemed to have addressed with alternative design details. Typical designs from that period included drains being associated with control joints placed above concrete keys, trenched into the foundation. The individual associated with the original design (interviewed by the IFT) recalled the design of this type of keys (which never made it into the bid specifications), but did not recall details related to placing drains beneath the chute slab.

### 2.1.2 Foundation Cutoffs

The service spillway does not have any foundation keys or cutoffs. These would have been excavated laterally, the width of the chute slab, below the slab base. They would be used to key the slab into the foundation rock. The keys could prevent sliding of sections of the chute slab where the foundation is weak, and helps cutoff seepage flows in the foundation. Typical details, shown in Appendix E, would allow for a more robust key between upstream and downstream slabs, would include a lateral drain on the upstream side, and would limit the movement of water in the foundation, and the slab section above the key. The lack of these keys at Oroville may have played a secondary role in the failure, because slab sections on weak foundation could slide downstream until they are supported be slab sections on stronger foundation. This could allow greater opening of cracks and joints in the slab. This sliding and joint opening may have resulted in the observed joint openings in the chute above the section that failed on February 7, 2017 (see Appendix I). The individual associated with the original design that the IFT interviewed thought that keys were included in the preliminary design. However, there were no keys shown in the bid specifications. Later, during construction (see the spillway timeline at the end of this appendix) the need for foundation keys was raised, but there was no follow-up design indicated in any of the documentation reviewed by the IFT. It is thought that keys were not included over concern that blasting for a key trench would lead to unnecessary foundation damage. Apparently no other methods for excavating these keys was considered.

### 2.1.3 Chute Slab

Design calculations provided by DWR for the spillway did not include any calculations or design criteria for the spillway chute slab. The IFT's understanding of details of the spillway chute slab design are based on the specifications paragraphs and drawings. The chute slab was designed to be a minimum of 15 inches thick, with a single mat of reinforcement near the top face (Figures A-4 and A-6), consisting of No. 5 bars at 12-inch spacing each way. The clear cover for all reinforcement in the chute was 3 inches. The concrete was designed for a 28-day compressive strength of 3,000 psi, and the reinforcement was 40 grade steel bars. Type II cement was used for the concrete, and the coarse aggregate was up to 6 inches in diameter. While large (6-inch) coarse aggregate can be beneficial in thicker placements, it would seem to be too large in a 15-inch slab with 3-inch rebar cover and 7-inch drain cover. Aggregate of that size has generally only been used for mass concrete placements. The construction documentation also indicates that some of this aggregate caused breakage of the herringbone drain pipe as concrete was being placed.

According to the specifications, the chute concrete was to be placed on a foundation consisting of moderately weathered rock or better, which was to be pressure-washed to remove all mud, debris, and loose or unsound rock fragments. There appears to have been no difference in specified foundation preparation among the gated headworks structure, the chute walls, and the chute slab. These specification requirements would have resulted in a concrete slab that was well bonded to the foundation below, assuming that the moderately weathered rock could be properly pressure washed to provide a bonding surface.

The as-built changes resulted in the herringbone drains embedded 7 inches below the flow surface. This results in a reduced cross section at the drain locations. This reduced cross section can be expected to be a stress concentration point, and designers typically would have provided additional reinforcement in this area to prevent excessive cracking and crack opening. However, this provision was not part of the design, and there was no room for an additional mat of reinforcement to be placed over the drains. Therefore, the bar spacing would have needed to be reduced to provide additional reinforcement. The lightly reinforced slab, coupled with the design joint spacing of 40 feet in the transverse (left to right) direction and 50 feet in the longitudinal (upstream to downstream) direction would likely result in intermediate cracking of the slab, even if it were well bonded to the foundation rock.

The seasonal temperature stresses on the hardened concrete slab compound the cracking issues. The specification paragraphs [A-2] for "Maximum Temperature of Concrete" states "the temperature of concrete at placement shall be as close to 50 degrees F as can be obtained by the following means…" Crushed ice up to the full amount of mixing water was recommended. The paragraphs for concrete mixing required that the mixing water not be added until arrival at the forms when the concrete temperatures could be 75 degrees or higher, and enough ice should be added to keep the temperature of the placed concrete below 75 degrees. Considering the southwest facing orientation of the spillway chute, the concrete could be expected to have a high set temperature in summertime conditions, even with these precautions. This could result in significant cracking and crack opening during winter temperature conditions, given the light, single mat of reinforcement provided in the design.

### 2.1.4 Slab Joints

There has been considerable confusion about the types of joints used in the spillway chute slab. Expansion joints were shown in the bid specification drawings, and contraction joints were shown in the as-built drawings. Both types of joint have no bond or bonded reinforcement across the joint. The difference is that expansion joints have a compressible filler material in the joint that allows the joint to close under high temperature stress, while the contraction joint has a painted or sprayed on bond breaker to prevent bonding of adjacent concrete placements. The bid specifications and the as-built drawings were ambiguous and unclear about the location of these joints. The Plan and Profile views on as-built Drawing A-3B9-2 do not clearly state that the joints being shown are the expansion joints for the wall stem and base only, and do not apply to the chute slab. However, Section C-C, cut on the plan view of that drawing and shown on as-built Drawing A-3B9-4, shows that the chute invert has a contraction joint at the same location that the walls have expansion joints. It is not clear if the contraction joint detail shown in Section C-C for the joints at Sta. 33+00 and Sta. 37+00 are intended to be applied to all lateral chute slab joints, or to some of the lateral joints, but no other lateral joint details are provided for the spillway chute (Figure A-10). Part of this confusion may be due to the bid specifications drawings (Figure A-2) having expansion joints in the chute slab. There is no indication why the expansion joints were removed, but as discussed in Appendix E, expansion joints on high velocity flow surfaces were not considered good practice at the time (and still are not today).

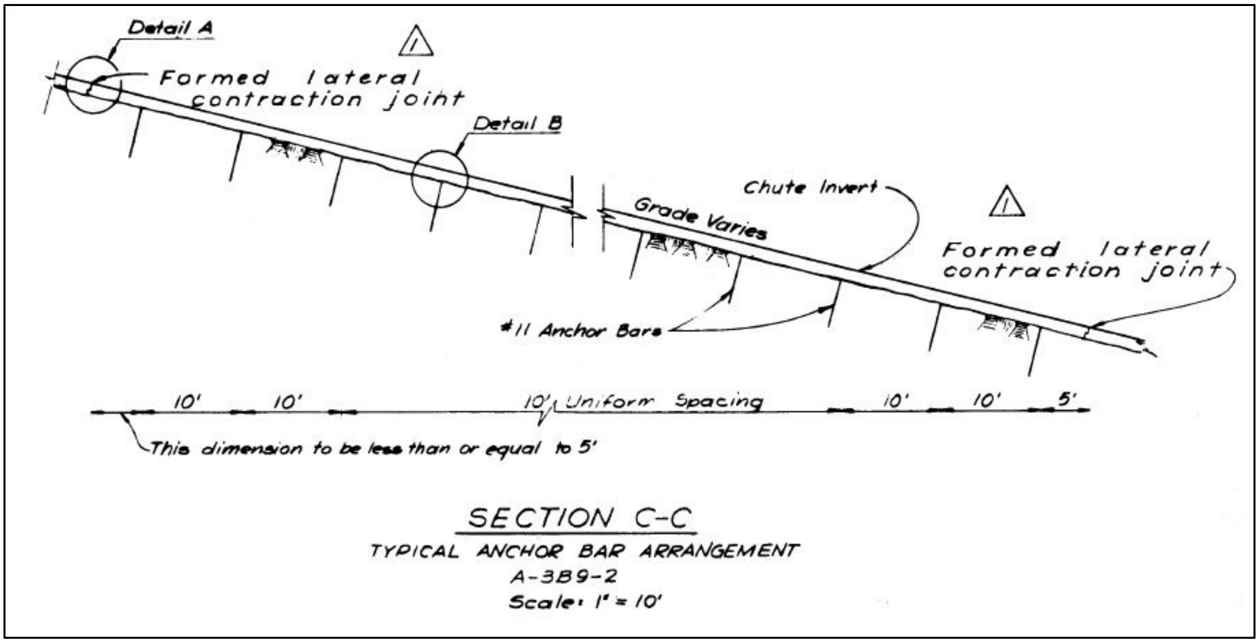


**Figure A-10: Spillway Chute Anchor Bar Arrangement Showing Transverse Invert Contraction Joints (from Drawing A-3B9-4)**

The chute slab lateral (or transverse) contraction joint details include a 7½-inch deep key where the downstream edge of the upstream slab overlaps the upstream edge of the downstream slab. The detail includes a ½-inch offset that is lower on the downstream side, but transitions back to the normal chute slope within 6 inches of the joint. This detail was observed in the field during the IFT site visit at some locations, but it is difficult to tell how frequently it was used due to surface erosion of the concrete making a ½-inch offset difficult to observe at some of the lateral joints.

At some locations on the drawings, the lateral contraction joints were depicted without these offsets. Construction records and photographs indicate that much of the chute slab was constructed using slip-form methods. It is believed that the slip-forming used to construct the chute concrete allowed the contractor to place more than one 50-foot long concrete chute panel at a time. It is further believed that the contractor "cut" shallow grooves in the freshly placed concrete to initiate a crack at intermediate (50-foot) locations within the slip-form length, rather than stopping the slip-forming operation to construct a formed contraction joint (see Spillway Chute Construction Timeline in this Appendix). All contraction joints were to be treated with 1-inch deep by ¼-inch wide filler material, placed in a groove formed at the top of the joint. Corners of the joint were to be tooled to a round edge (see Figure A-11 for joint details).

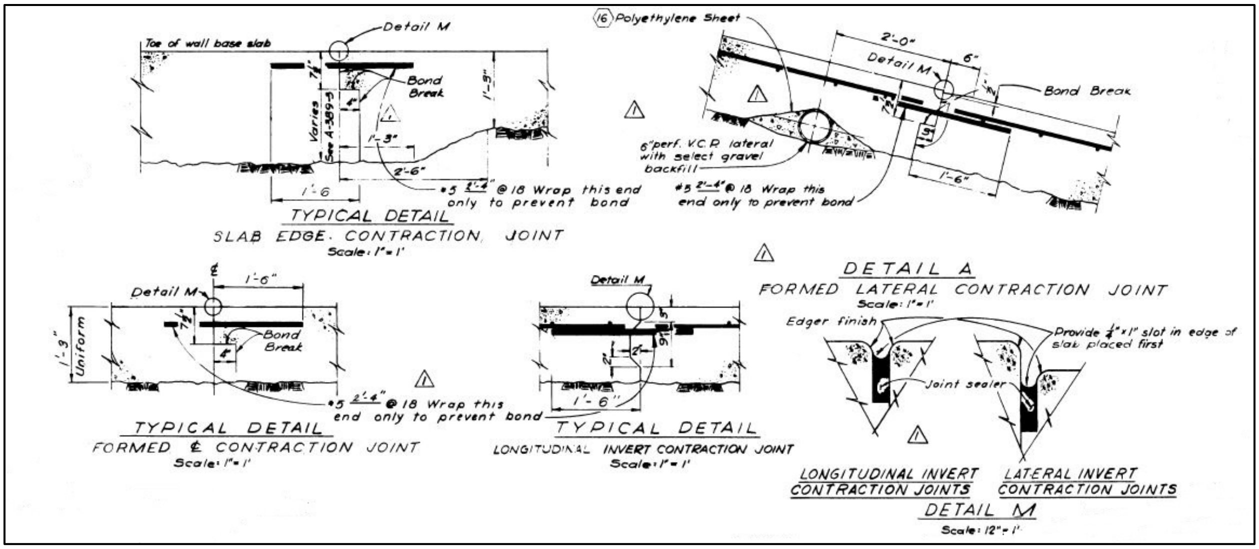


**Figure A-11: Chute Slab Joint Details (from as-built Drawing A-3B9-4)**

Concrete in the spillway chute was constructed in six longitudinal lanes, numbered from right to left looking downstream. Lanes 1 and 6 were for the wall bases, while lanes 2 through 5 were for the chute slab. The chute slab lanes were 40 feet wide, although as-built Drawing A-3B9-2 shows eight 20-foot wide lanes, rather than the actual 40-foot wide lanes. It appears that the 40-foot lane configuration was a change that occurred after the bid specifications drawings were completed, and this change was not reflected in the as-built drawing revisions of September 30, 1968.

Three different as-built longitudinal contraction joint details were employed (Figure A-11). The joint between the wall bases in lanes 1 and 6, and the adjacent chute slab placements include a 7½-inch deep by 4-inch wide key, blocked out of the wall base so that the slab placement overlaps the wall base (presumably the wall base was to be placed first). The centerline joint has a blockout like the wall-slab blockout, but it is not clear from the drawings which side of centerline was to have the blockout. The intermediate joints (left and right of centerline) are keyed at mid-height of the slab, 2 inches deep and 5 inches wide with 45 degree sloping sides, making it 9 inches wide at the maximum width. Bond break is called out for the joint surfaces of the wall edge and centerline joints, but not for the intermediate joints. However, visible evidence of unbonded intermediate joints in the field based on post-incident photos and video indicates that these joints may have also been treated with bond break.

No. 5 dowel bars at 18-inch spacing cross each of the chute slab contraction joints. These bars are embedded and bonded for a length of 18 inches on one side of the joint, and wrapped to prevent bond for 10 inches on the other side. These bars are shown to be placed just below the reinforcement mat at the top of the slab. With the dowels and rebar mat both being placed near the surface (as compared to dowels being placed at mid-height) there is an apparent plane of weakness created by the reinforcement near the joint, that could have increased the potential for delamination and spalling of the concrete near the joints.

Waterstops were generally not provided in the chute slab joints. The only locations where waterstops were provided is between the chute slab and the headworks at the upstream, and between the chute blocks at the downstream end. As discussed in Appendix E, waterstops were not commonly used in chute slabs when Oroville Dam was constructed.

### 2.1.5 Foundation Anchors

The service spillway chute slab design included No. 11 anchor bars spaced at 10 feet each way along the foundation surface (see Figures A-12 and -13). The anchor bars were intended to resist 5 feet of uplift [A-8]. The designed embedment lengths were likely insufficient to develop the full design tensile strength of these bars based on the assumed foundation conditions. The design embedment of the standard 90-degree hooks into the concrete slab is only 10¾ inches. The ACI-318 Code from that period would have required about a 21-inch embedment. While it is understandable that the designers would not have wanted to make the slab 10 inches thicker to accommodate the No. 11 hook embedment, a typical design might call for smaller bars more closely spaced to achieve the desired pull-out strength. For example, the designers could have used No. 8 bars spaced at 6 feet each way. However, even with No. 8 hooked bars, embedments could not fully develop the hook strength in the concrete, so it is possible that the anchors were oversized to provide sacrificial steel to accommodate possible corrosion. A 4-foot square piece of welded wire fabric was to be placed at the base of the concrete slab where the anchor bars were installed. Presumably this was to help prevent pull-out of the bar from the concrete.

The anchor bars were to be embedded 5 feet into the foundation in holes that were specified to be 1½ times the bar diameter, with the bars to be encased in cement grout. These bars were to be tested in the lowest quality rock. No strength requirements were provided in the specifications. Post construction documentation indicates they were to resist 5 feet of uplift.

There were no specification provisions for overexcavation with respect to anchor bar installation. Based on Figures A-12 and A-13, it is not clear whether the bars were lengthened to maintain 5-foot foundation embedment, the foundation embedment was reduced to keep the top of the anchor bar hook at its design location just below the rebar mat, or if the bars were all adjusted to maintain the specified embedments in both the slab and foundation while keeping the bar lengths constant. It is also not clear what rock would have been considered “sound rock” for the referenced 5-foot embedment into such rock. Two observations from the construction photos would tend to indicate that the required embedments in the foundation were either not always maintained, or that longer bars were sometimes used to compensate for overexcavation. Many photos show the tops of the bars just below the rebar mat as designed, even with a varying foundation surface (Figure A-13). Photos of the downstream area near the end of the chute show bars that appear to be longer and that have 180-degree hooks (see Figure A-9).

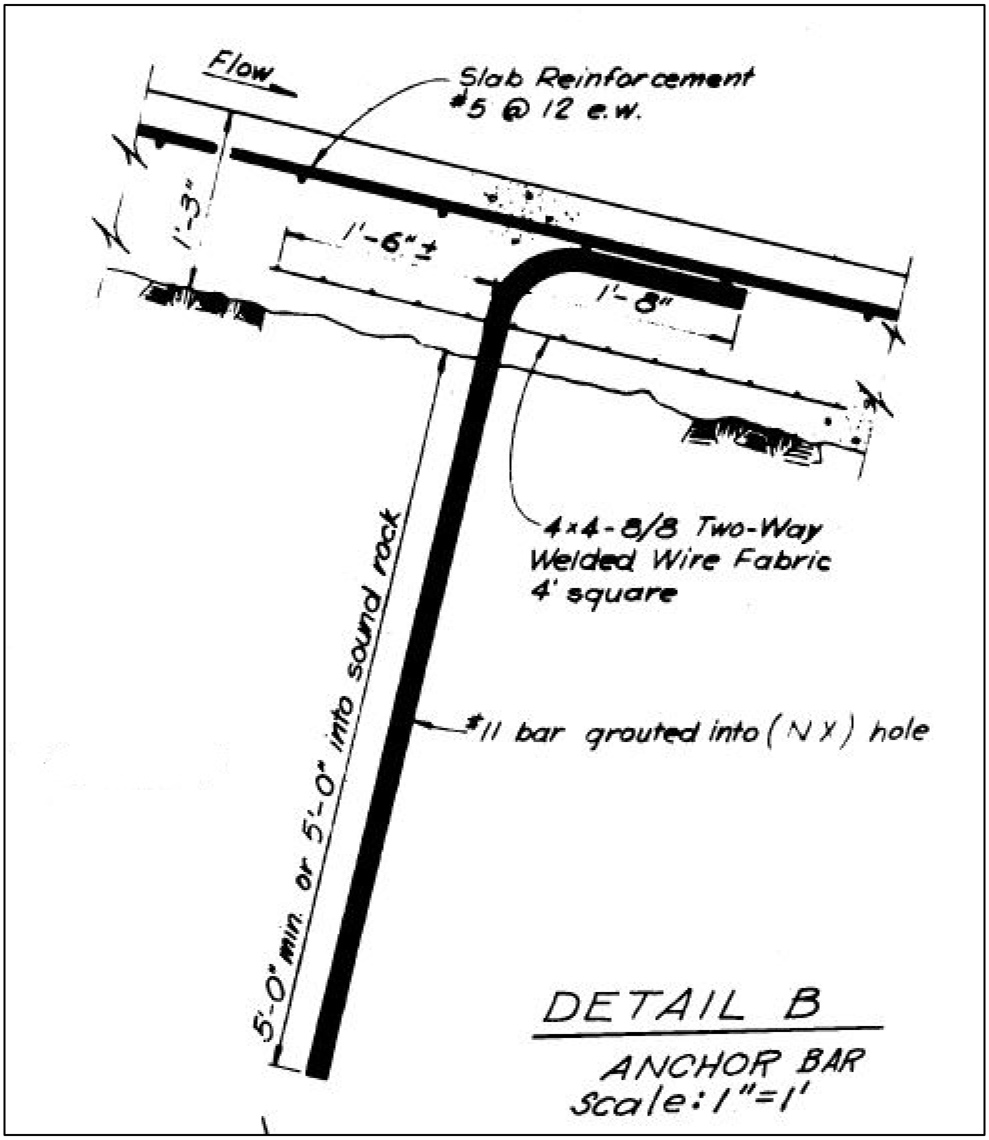


**Figure A-12: Anchor Bar Detail (from Drawing A-3B9-4)**

As with the chute concrete, no details of the anchor bar designs for the spillway chute were provided, and it is possible that none exist. Therefore, it is not known why the designers chose to install anchor bars in the spillway chute. Although no design calculations were provided for the spillway chute, the wall design calculations had load cases where there was water both inside the chute (up to the PMF condition), and outside the chute. These water loads were used for the design of the reinforced concrete wall stem and base. Although stability calculations were also performed, it appears that uplift was not generally considered in the chute wall calculations, except for the walls at the terminal structure at the downstream end. The design of anchor bars for the chute walls was not documented in the design calculations provided by DWR.


**Figure A-13: Anchor Bars Installed on Uneven Rock Surface (Photo OD_13231_10_06_1966)**

A typical anchor bar design for a spillway chute would include an analysis for floatation. This load case would apply to gated spillways where flow could be shut off suddenly. In this calculation, the water level in the chute is assumed to apply to the foundation as well. The uplift beneath the chute would be assumed to be the height of water in the chute plus the thickness of the slab. If the flow were suddenly shut off, the weight of water in the chute would be removed, but the uplift beneath the slab would remain temporarily. The resulting floatation would be resisted by anchor bars that anchor the slab to the foundation. No such analysis could be found for the Oroville Dam service spillway.

## 2.2 Emergency Spillway Design Details

The following discussion addresses three aspects of the emergency spillway design, namely structural stability, erosion protection, and downstream erosion.

### 2.2.1 Structural Stability

The stability analysis of the gravity ogee weir section of the emergency spillway crest structure was considered in the spillway design calculations [A-9]. This analysis indicated that the uplift pressure (equal to full reservoir) at the heel (upstream edge) of the crest structure exceeds the computed foundation pressure produced by the loads when the reservoir is at the top of the weir (Load Case 1). At the design maximum water surface for the probable maximum flood (PMF) condition, water surface elevation 917 (Load Case 3), the uplift pressure is 4,000 lb/ft$^2$ above the foundation pressure of 180 lb/ft$^2$ (only 3,000 lb/ft$^2$ above the base pressure by their calculations – see below). Based on the Bureau of Reclamation's *Design of Small Dams*, [A-10], which had been available since 1960, when a rigid body analysis is performed and the uplift pressure at the heel (upstream edge of the foundation contact) of a gravity section exceeds the foundation bearing pressure plus the tensile strength of the concrete/foundation contact at that location, a cracked section analysis should be performed. The foundation contact strength would need to be 4,000 lb/ft$^2$, or about 28 psi for stability of this structure. This requirement apparently was not checked. While it may be reasonable to expect some cohesion at the contact, the cohesion required for stability could significantly increase if the toe of the gravity section is undermined. It is not clear if this potential stability issue was ever considered prior to the initial operation of the emergency spillway.

In computing the "sliding factor," which is the required friction factor to prevent sliding, the computed uplift was not included in the calculation. If uplift had been included it would reduce the foundation contact pressure such that the required sliding factor for Load Case 1 would be 0.67 rather than the computed 0.43. For the PMF, Load Case 3 the uplift was not increased by 17 feet over load Case 1 as it should have been. Given this increase the sliding factor would need to be 1.18, which translates to a friction angle of about 50 degrees. Given the potential for cohesion this correction would still indicate the foundation is stable if the contact tensile strength is greater than 28 psi. However, the original analysis shows that the designers were not using state-of-the-practice analysis methods from the 1960s, and as a result, may have overstated the stability of this section. The analysis should be corrected for use in future failure mode analyses.

Sliding stability is improved by drainage provided beneath the mass concrete weir. However, it does not appear that there is any way to maintain this drain, and it would only collect seepage in a limited area where joints are in contact with the narrow drain. The drawings do not show any drilled drains into the foundation that could be maintained over the life of the structure. Given this configuration, it is believed that the 50 percent drain efficiency assigned to this drain is overstated, and sliding factors of safety may be lower than have been calculated.

### 2.2.2 Erosion Protection

The overflow weir was designed with an 11.67-foot long downstream apron to protect against erosion as flows over the ogee weir reach the foundation elevation (see Figure a-14). While this apron will absorb some of the impact of direct flow onto the foundation, it does not protect against downstream erosion and headcutting. Spillways with this configuration have experienced significant erosion holes downstream of the apron, which could also undermine and fail the apron, if foundation conditions are unfavorable. Additionally, a large erosion hole could remove all passive resistance to sliding of the gravity structure, if any foundation features daylight into the erosion hole.

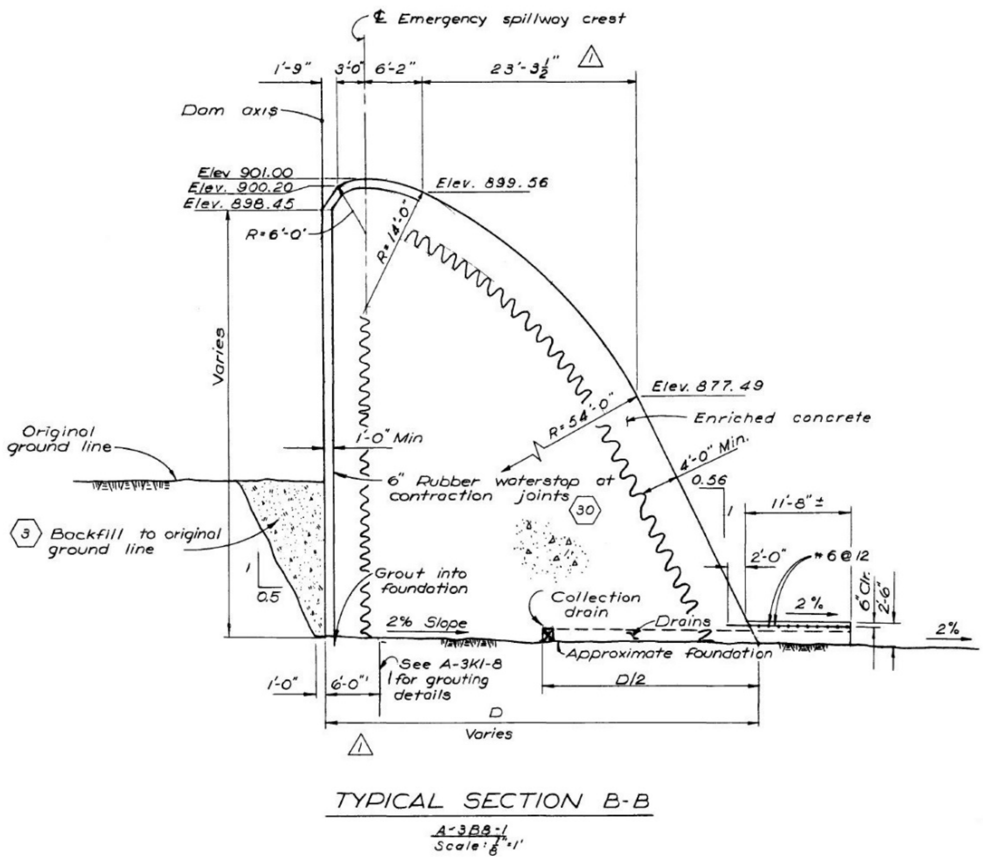


**Figure A-14: Ogee section of the emergency spillway showing the downstream apron**

### 2.2.3 Downstream Erosion

The downstream rock quality and erodibility is discussed in Appendix C. At the time of the original design, tools and methods such as stream power erosion analysis were not available to the designers. However, if the rock downstream was thought to be erodible, more protection would have been required than just the short apron provided.

## 2.3 IFT Findings Related to Design

The IFT noted several issues related to the design of the spillways that may have contributed to the problems experienced during the 2017 flooding. Appendix E specifically addresses how these designs compare to other designs over a 20-year period around the time Oroville spillways were designed. Design issues are discussed here:

- The foundation drainage system lacks the redundancy of intermediate longitudinal collector drains. It relies on all flows being collected on one side of the spillway chute or the other, where surface runoff is also collected. Any plugging of either a collector drains, or the individual herringbone drains could cause backups in the drainage system.
- The design of the herringbone drains to be placed within the chute slab thickness resulted a reduce concrete section over the drains where cracking could occur. Since the drains were

only designed to collect groundwater seepage, flow into cracks in the concrete chute slab could potentially exceed the drain capacity.

- Concrete cutoffs beneath the chute slab, had they been installed, could have helped minimize slab movement in areas of weaker foundation material.
- The chute slab thickness of 15 inches seems to be thin for a spillway chute on one of the tallest dams in the United States. However, the single layer of light reinforcement and the reduced thickness at drains are of greater concern.
- The keys at slab joints, particularly the transverse joints could have been more robust if they were coupled with foundation cutoffs. However, the biggest problem at the slab joints is the lack of waterstops, which were generally not added to chute slab designs until Oroville Dam spillway was constructed.
- The foundation anchor design strength was not well documented in the bid specifications. Embedment lengths into the foundation were to be tested in the field. At the time Oroville Dam was designed, chute anchors bars were typically only being designed for minor uplift pressures.
- The emergency spillway crest does not seem to have been adequately designed for uplift and tension at the heel. Stability at maximum reservoir during a PMF will depend on bond strength between the concrete base and the foundation.
- The short apron downstream of the emergency spillway crest does not provide any energy dissipation, but does direct the flow in a horizontal direction over the downstream rock. However, flow velocities at this location are still high. It is apparent that the potential for rock erosion in the unlined downstream channel was not understood by the designers.

## 3.0 CONSTRUCTION OF THE SERVICE SPILLWAY

## 3.1 <u>Foundation</u>

Foundation geology is discussed in Appendix C. The forensic investigations of the spillway chute and foundation are discussed in Appendix D.

The foundation was excavated to grade using blasting and heavy equipment. Smaller equipment and hand excavation was used to achieve the final grade. Pressure washing using a water-air jet was used for final cleanup.

### 3.1.1 Cleanup and Preparation for Concrete Placement

According to the specifications [A-11] (Specifications No. 65-09, Section 17(m)(2)), the concrete was to be placed on a foundation consisting of moderately weathered rock or better, which was pressure washed to remove all mud, debris, and loose or unsound rock fragments. There was no distinction in the specifications between the level of effort to clean-up the headworks structure, emergency spillway crest structure, or spillway chute structure foundations. The only difference found was that sharp points in the chute foundation could project 3 inches into the limits of concrete, as shown on the drawings (Specifications No. 65-09, Section 14(d)(5)). Appendix C includes a detailed discussion of the specified foundation requirements, and what transpired during construction, including discussions related to Change Order No. 21.

The discussion in Appendix C and the Final Construction Report [A-4], with regard to foundation overexcavation, indicates that there was a disagreement between DWR and the contractor about how much overexcavation was required. DWR held the position that overexcavation was only required at shears zones, and other than clay seams, they did not direct the contractor to overexcavate the foundation. From daily reports, there was an ongoing struggle to achieve adequate foundation (see the Timeline section of this appendix). At some point during the construction it was apparently decided that the requirements noted in the previous paragraph did not apply to the spillway chute, and the requirements seem to have been relaxed. An observation of an acceptable foundation described as showing 50 percent rock is mentioned in the September 21, 1966 DSOD Inspection Report of an August 29 inspection, when an area of foundation between Sta. 26+00 and 27+00 was inspected. While the DSOD inspector seems to indicate better cleanup of the foundation is needed, his description of 50 percent rock showing seems to be taken as the standard being used by the construction staff.

Final cleanup was to be achieved using a pressurized water-air jet. It is believed this would have been the same or similar to the method used for the headworks and emergency spillway crest structure foundations. However, the same level of effort does not appear to have been used, since only 50 percent of the foundation was required to be exposed rock. In some locations, the contractor tried to clean up the foundation using just air, because there was very little rock and the water turned the material into mud (see the Timeline below).

In some construction photos, it appears that not even this requirement was met in areas of highly weathered foundation, where it seems that large areas of the foundation did not even meet the 50 percent requirement. It seems that DWR staff believed that this condition was acceptable if adjacent areas had more exposed rock. Figure A-15 shows the foundation cleanup just before placement of concrete in an emergency spillway monolith. The foundation was washed and there was no loose material present. The cleanup resulted in a foundation that is not very uniform in elevation. This photo seems to represent the effort of cleanup required by the specifications paragraphs. Figures A-27 through A-29 show foundations after cleanup in the spillway chute. In some cases, it appears that there is less than 50 percent intact rock showing. A good portion (greater than 50 percent) of these foundation areas appear to be rock fragments and soil. In Figure A-17 the fines seem to be washed off the rock, but that does not seem to be the case in the other two figures.

Engineer's Daily Reports indicate greater concern with the contractor’s cleanup efforts, as the construction progressed towards the area of the chute where the 2017 failure initiated. More than one daily report indicated cleanup problems between Sta. 32+00 and 33+00. Based on the daily reports, it is believed that the foundation in Figure A-16 is an area that required rewashing, and conditions did not improve downstream from Sta. 33+00, where the rock quality was poor.

According to the Final Construction Report, “In the chute there was very little extra excavation directed. This consisted of a few clay seams in the foundation and the areas where the slope failure occurred.”

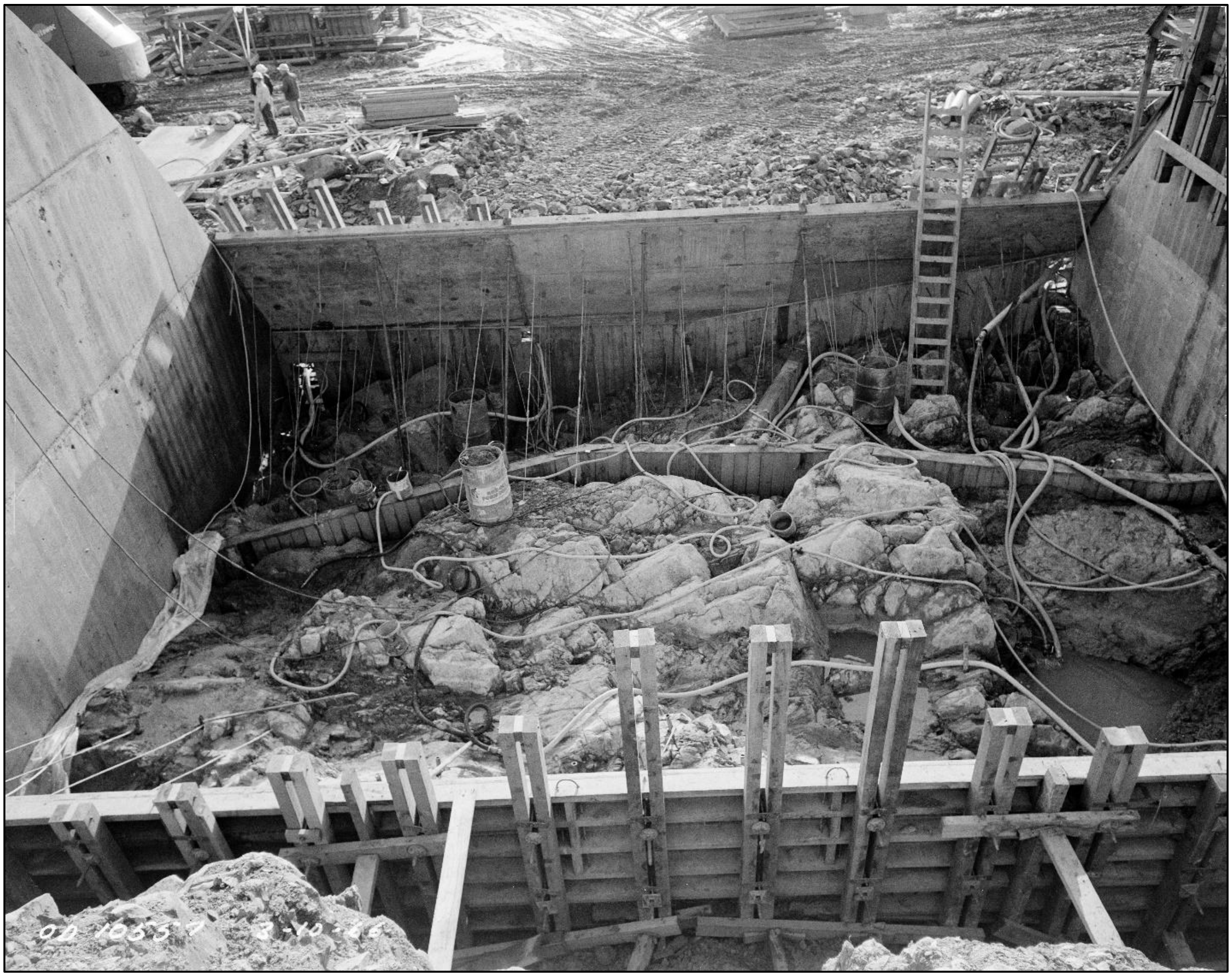


**Figure A-15: Clean-up effort for the emergency spillway gravity crest structure just prior to placement. Note that all mud, debris, and loose or unsound rock fragments have been removed and the rock has been pressure washed. (Photo OD_10557_03_10_1966)**


**Figure A-16: Clean-up effort for the spillway chute prior to placement. Note that foot prints appear in the material left in place, and numerous rock fragments are present. This is believed to be in an area that required additional cleanup just upstream from Sta. 33+00. (Photo OD_13420_10_26_1966)**


**Figure A-17: Another example of the clean-up effort for the spillway chute prior to placement. Note what appears to be "compacted clayey fines" between the rock outcrops. (Photo OD_13484_11_10_1966)**

### 3.1.2 Foundation Anchors

Three sets of holes were drilled in the chute foundation for testing the No. 11 anchor bar strength. This included two anchors at each of three trial embedment lengths: 5 feet, 6 feet, and 7 feet. Anchors for each length were installed in average foundation conditions and in "worst case" conditions that included clay seams. The specifications were not clear on the strength requirements for the anchor tests. This was determined to be 30,000 psi during construction according to the Final Construction Report. However, an Engineer's Daily Report from September 6, 1966 indicated the tests were made when the contractor was working between Sta. 26+00 and 28+00, and that all anchors passed a pull test of "30,000 pounds," which is different from the requirement of 30,000 psi for No. 11 bars. It is not clear if there was an error in the write-up or if the test was conducted for a lower strength than required. However, if the anchors were designed to resist 5 feet of uplift, the required strength of 30,000 pounds makes more sense. With an nominal yield strength of 40,000 psi, neither requirement would have developed the full strength of the bar. It is assumed that the bars were oversized, possibly to add greater bar area to counteract minor erosion. Figure A-18 is believed to show the area near where the anchor bar pull tests were performed (10 days after the pull tests), based on the date of the photo compared to the construction activities as reported in the daily reports from construction. Note that in Figure A-18 it would be difficult to determine where there were worst case conditions having clay seams, unless there was a special cleanup for the anchor bar pull tests.

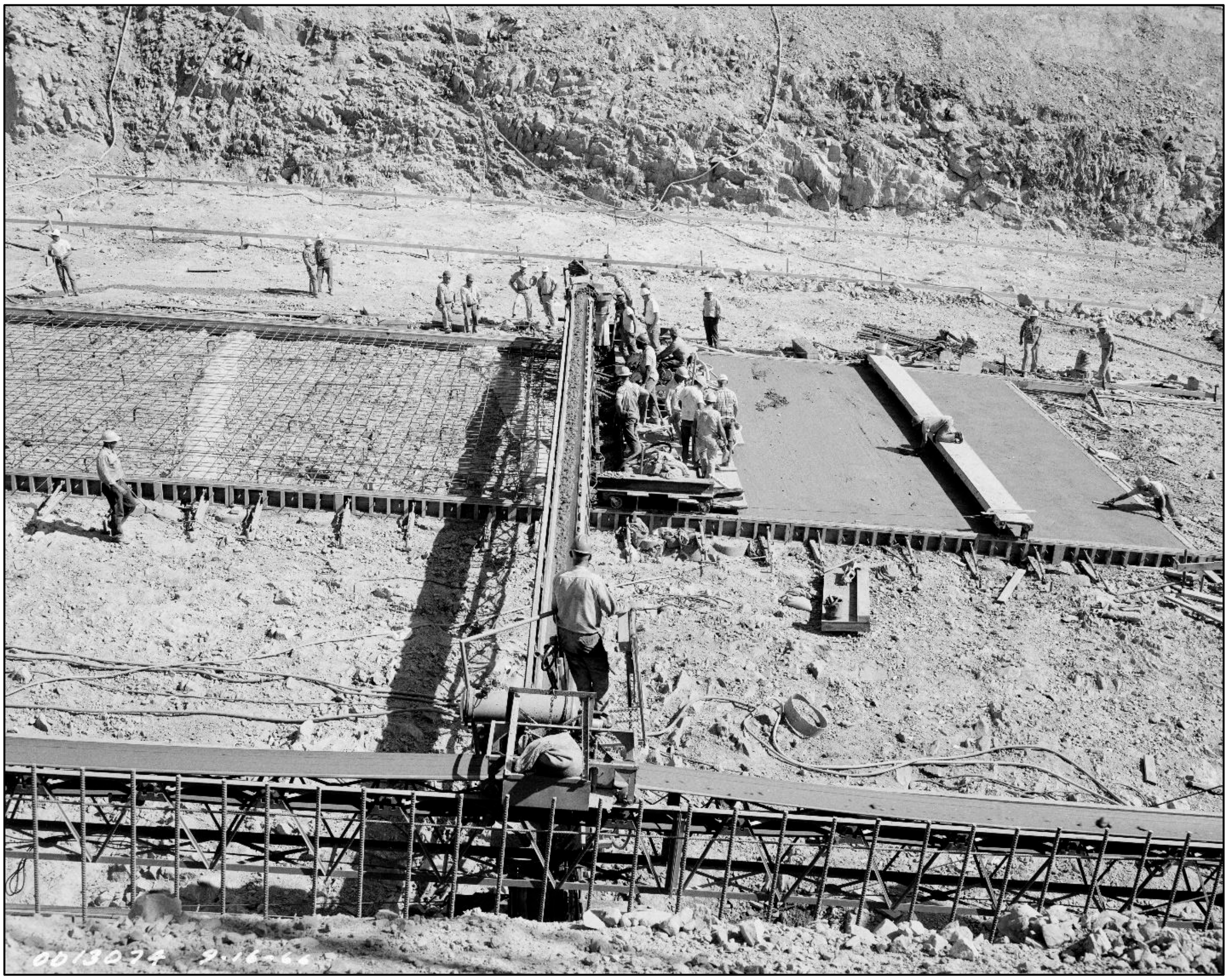

**Figure A-18: Typical concrete placement in the spillway chute (believed to be Lane 3, Sta. 26+00 to 27+00) using slip-forming. Note that the foundation ahead of the slip-form shows very few distinct rocky outcrops. (Photo OD_13074_09_16_1966)**

The minimum embedment length was determined to be 5 feet. However, it is not believed that this embedment would have been adequate for foundation conditions encountered in the worst locations of the spillway chute excavation where the initial failure occurred, which were yet to be uncovered at the time of the anchor bar pull tests. Shearing and weathering near Station 33+00 appears to be more pronounced than between Station 26+00 and 27+00 (See figures in showing geologic mapping in Appendix C and D). This point was proven after the 2017 damage occurred, as anchor bars were still attached to the concrete slab in some places, where they either completely pulled out of the foundation, or the foundation crumbled around the bars and was washed away (see Figure A-19).

In Figure A-19, the underside of some of the slabs appears to be smooth, indicating that the slab was founded on compacted soil-like material rather than an irregular excavated rock surfaces. The fractured rock in the background of this figure may be somewhat representative of the foundation beneath the exposed slab. There also appears to be a gap at the foundation/rock contact where intact foundation remains, indicating that the “compacted clayey fines” or soil covering may have been extensive. Soil covering the foundation would have been critical at this location, because it would obscure the highly fractured nature of the rock below. Without this knowledge, there would be no way to determine if the anchor bar embedment lengths needed to be extended further into the foundation, due to the conditions being worse than that in the area where embedment lengths were tested.

**Figure A-19: Anchor bars on the underside of the slab upstream of Sta. 33+00 (on right edge). Note the smooth underside of the chute slab that was to have been placed on a rock foundation which would have been expected to be irregular. (Photo DSCF9788)**

Based on the October 26, 1966 Engineer’s Daily Report, anchor bars installed in Lane 3 from Sta. 33+00 to 34+00 had to be cut off and reinstalled because they were not straight. The report indicates that eight or nine anchors had to be cut off. In Figure A-20, taken on February 8, 2017, the cut bars can be seen alongside of the final anchor bars that pulled out of the concrete. From the figure, it appears that some of the replacement anchor bars pulled out of the concrete, while others apparently pulled out of the foundation. In Figure A-21, the same location can be seen being prepared for placement.


**Figure A-20: Damage in Lane 3 downstream from Sta. 33+00. Cut anchor bars are circled. These were replaced because they were tilted. Replacement anchor bars can be seen in two locations. Note the drain discharging from the wall at approximate Sta. 33+60. (Photo DSCF5904r)**

In Figure A-21 the drain pipe appears to be placed a considerable distance above the foundation, on a high mound of gravel that is contained by wooden forms. This is an indication that the foundation was overexcavated in this area (see Figure A-18 for comparison). It is not clear how, if at all, the anchor bars were adjusted for the overexcavation. From Figure A-20, the anchor bars that remain intact appear to be adequately embedded in the foundation. It cannot be determined from the construction photo (Figure A-21) if the foundation embedment was adjusted to the excavated surface or not. If not, some anchors may have pulled out due to inadequate embedment. In Figure A-22 it also appears that the remaining anchor bars (including 3 pairs of bars with one of the pairs apparently cut) may have been adequately embedded in good rock. Comments in the

Engineer's Daily Report from October 18, 1966 (see Timeline below) would also support the idea that the anchors were adjusted in the field.

It is difficult to interpret the exposed foundation in Figures A-20 and A-22, with the way the rock is broken, as being the same material seen in Figure A-21. One might expect to see a blockier surface in Figure A-21 if the foundation was excavated as expected with heavy equipment to remove weaker weathered material, and then washed to expose the fresh rock. If anchor bars were embedded 5 feet into the excavated surface seen in Figure A-21, it is easy to imagine that some of the holes were partially drilled through weaker surface deposits that were left in place due to the relaxed cleanup effort.

Anchor bars that can be seen in the foreground in Figure A-17 appear to be embedded in both hard rock outcrops on the surface and the "compacted clayey fines" between the outcrops. This photo is believed to be an area downstream from the initial damage (Lane 4, Sta. 35+00 to 37+00). Note that in this photo the anchor bars seem to be aligned vertically such that the 90-degree hook is just below the rebar mat. In this photo, intermediate straight anchors can be seen spaced between the anchor bars. These straight anchors were installed to support the rebar mat and probably are not embedded very deep in the rock. It appears that the No. 11 anchor bars may also be supporting the rebar mat.

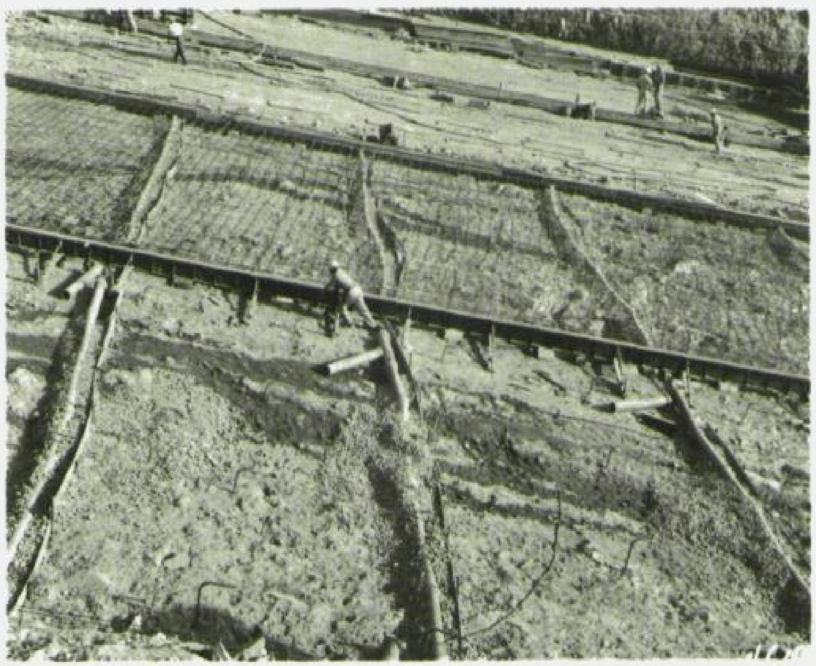

Photo 39. Chute foundation in vicinity of Sta. 33+60.
Tile and gravel underdrains in lanes 2 and 3, rebar in
lane 3. View southeast.
Neg. No. 4644 11-2-66

**Figure A-21: Lane 3 downstream from Sta. 33+00 being prepared for placement. Note that in both Lane 3 and Lane 2 (foreground) the gravel around the drain pipes was contained by wooden forms.**

Figure A-16, which is believed to be just upstream from the initial 2017 damage area also shows poor cleanup effort. The poor cleanup effort is also mentioned in the Engineer's Daily Report on the day after the photo was taken, presumably when this foundation was being accepted for placement. However, at the time the photo was taken, the anchor bars had already been installed.

**Figure A-22: Lane 3 downstream from Sta. 33+00 showing 3 sets of double anchor bars where one of the pair has been cut off. From this photo, it appears the bars at this location may have been adequately embedded in rock. (Photo DSCF5859)**

## 3.2 Drainage

The drains were increased in size after the bid specifications were completed (as discussed in the design portion of this appendix). The 6-inch herringbone drain alignments were to be field located depending on the foundation excavation conditions. The drains were to be placed within the specified slope tolerances for drainage purposes. Change Order No. 21 required that the herringbone drains be sloped 4 percent for drainage, and allowed for an additional 4½-inch overexcavation of the foundation. Figure A-21 shows typical drain installation prior to concrete placement. The herringbone drains were spaced at about 25 feet in the flatter, upper section of the spillway (Sta. 13+00 to 29+00) and about 20 feet in the steeper, lower section (Sta. 44+00 and higher).

Engineer's Daily Reports indicated that some of the 6-inch herringbone drains were broken when the concrete aggregate (up to 6 inches in diameter) was dropped from the conveyor belt directly onto the drain pipe. Presumably the incidents that were reported were observed by the construction inspectors, and the pipes were repaired. However, it is possible that some broken pipes were unnoticed and therefore not repaired or replaced. These drains were to be placed with the perforations facing the foundation. However, post-incident observations revealed that some pipes were placed with the perforations facing the top of the slab, and some without perforations at all (see Appendix D). HDR investigation included video inspections of drains in the upper chute found numerous issues with the herringbone drains [A-6]. In addition to issues with perforations, some drains were cracked, had open joints, drains sloping uphill against flow, and infilling of debris. Figure A-23 shows a cracked drain with sediment deposits. It is not known if the cracking happened because of construction activities, chute repair work, or from other causes. Figures A-24 and A-25 show some of the additional drain issues. While drains were not found to be plugged, the remaining upper chute drains may not be representative of the conditions where the chute failure initiated.


**Figure A-23: Herringbone drain inspection image. Note the drain has been cracked. (from HDR investigation [A-6])**

A common issue with the herringbone drains is that the drains were not placed directly on the foundation, but rather were placed well above the foundation when overexcavation occurred. The drains were intended to be placed in a gravel envelope that includes gravel below the drain pipe as well as over the top of the pipe. There was no filter material placed around the gravel to prevent fine foundation material or concrete from contaminating the gravel during construction. Polyethylene sheeting was to be placed over the pipe and gravel to protect it from the fresh concrete, which tended to isolate the drains from the cracks that formed above the drains. Fine-grained materials left on the foundation surface after cleanup could easily move through the gravel and into the drains due to lack of filtering. The gravel placement over the top of the pipe seems unnecessary, since the perforations were intended to be on the bottom of the pipe where they are less likely to be contaminated with concrete.

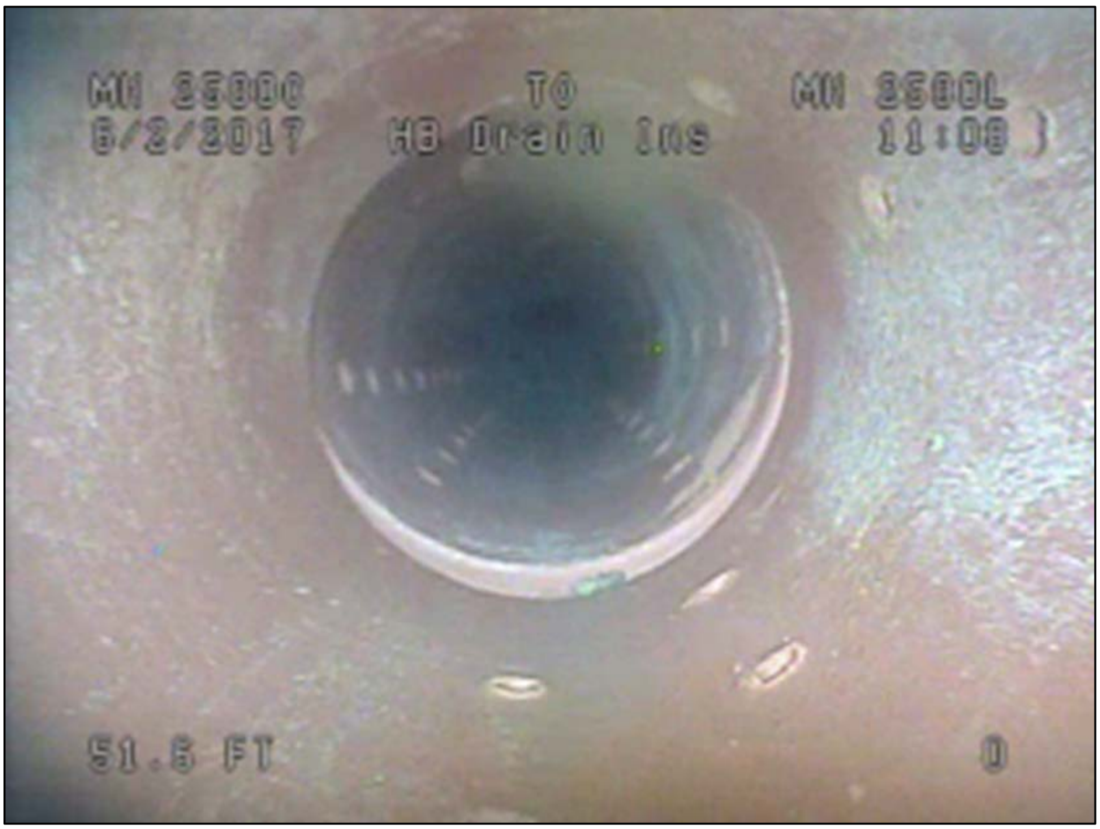


**Figure A-24: Herringbone drain inspection. Note that the perforations are rotated at the joint. (from HDR investigation)**

There is no indication that the gravel bedding was not placed beneath the pipes. In areas of overexcavation, it appears from construction photos that the concrete cover over the pipe was maintained at about 7 inches in most places. Several areas show evidence that the gravel was spread well beyond the drain locations, and it is not known if this was cleaned up prior to concrete placement. There were some locations observed during the post-failure investigations where the pipe seemed to be placed at the bottom of the slab where the slab section was thicker than the specified 15 inches, rather than 7 inches below the top of the slab (see Appendix D). Where overexcavation of more than several inches occurred, the gravel beneath the pipe was built up using wooden forms to contain the gravel. In other places where overexcavation was more than a couple of feet, the pipes were connected to the foundation below by placing gravel-filled sonotube risers between the herringbone drains and the foundation. The result of these adjustments for overexcavation is that the drains were placed within the concrete slab a foot or more above the foundation surface (Appendix D). These drains would have little chance of collecting surface flows at the foundation contact until the ground water backs up enough to enter the drains. Figures A-13 and A-26 show details of drain construction where there was significant overexcavation.

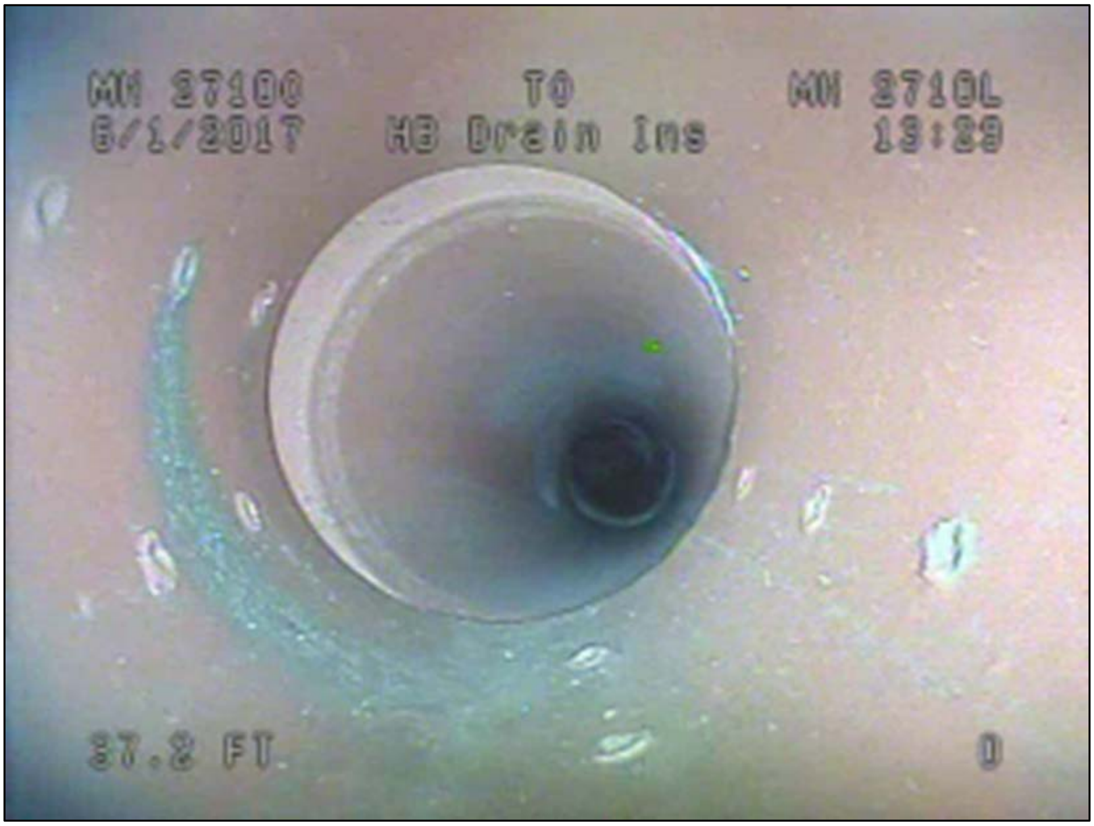


**Figure A-25: Herringbone drain inspection. Note that the section beyond the joint has no perforations. (from HDR investigation)**

For the drains constructed on top of gravel-filled sonotubes, it is unclear how they were constructed. If the drains were completely wrapped in plastic, as suggested in Figure A-26, there would be no way for them to connect to the foundation through the gravel columns. If the plastic was just over the top of the drains (as it was for more conventional installations), concrete could flow into the perforations. It appears from Figure A-26 that the drains would have been wrapped in plastic everywhere, except where they contacted the sonotubes. No matter how they were constructed, it is the IFT’s opinion that drains constructed in this manner would be relatively ineffective.


**Figure A-26: Herringbone drains being constructed on top of gravel filled sonotubes. The drains appear to be wrapped in plastic. Based on the worker standing beneath the drain pipe, the drains were likely placed up to 6 feet above the foundation they were intended to drain.**

The 6-inch herringbone drains discharge into a series of 12-inch collector drains. The collector drains were constructed in pairs, with each pair being constructed with similar details on the left and right sides of the spillway chute, outside of the chute walls. The collector drains as detailed on the as-built drawings are 12-inch pipes (Figure A-6). The 12-inch pipes were a modification required by Change Order No. 21. Separate systems were provided for each drainage zone. These zones were located in the field (Figure A-7).

Change Order 21 directed the contractor to make changes to the drainage system based on Consulting Board recommendations (see Appendix C). This required resloping and scaling of the spillway chute rock slopes affected by the revised drainage system. The pipe sizes were increased, and slopes adjusted for positive drainage. [A-3]

Perforated pipe was used in the upstream section of each collector drain system, which is the portion of the collector drain that is connected to the herringbone drains. The perforations in the collector drains were to be placed facing up, to collect seepage into the gravel fill above. The downstream portion of each collector drain consists of solid 12-inch pipe, which exits through the chute walls, such that the drains discharge into the chute. Figure A-20 shows a drain that is discharging into the chute. The perforated portion of the collector drain system generally follows the slope of the spillway chute in order to allow connection of the herringbone drains, after they pass under the chute wall base. The solid portion of the collector drain system was constructed at a flatter slope than the chute, in order for the drains to eventually exit back into the spillway chute near the top of the chute walls, downstream from the collection zone.

A vertical cleanout pipe is located at the upstream end of each separate collector drain section. It was intended that this cleanout be used to flush water down the drains to remove sediment and debris.

Each collection zone was constructed to cover varying lengths of spillway chute, and the number of herringbone drains each collector drain serviced also varied. Generally, if one of the collector drain solid outfall pipes were plugged, it would affect the entire zone. There was no redundancy in this collector drain system.

### 3.3 Chute Concrete

Chute invert concrete placements began on September 8, 1966. A conveyor system was used, along with a 40-foot wide steel beam slipform used to screed the concrete to its finished configuration (see Figure A-18). This slipform and conveyor system rode on two steel rails on each side of the chute. Based on photos and the Final Construction Report, this slipform system was primarily used on the steeper portion of the chute. A truck mounted crane was used to place concrete on flatter portions of the chute, and in the chute walls.

The spillway chute placements were made in lanes. There were six lanes that included the right and left wall base placements (lanes 1 and 6, respectively). Lanes were numbered right to left looking downstream. Lanes 2 through 5 were each 40 feet wide. Generally, 100 feet of chute (measured longitudinally) was placed at a time. However, some of the Engineer's Daily Reports indicated as much as 200 feet of slab was placed in a day by slipforming.

There were many places where overexcavation occurred. There is evidence that backfill concrete was placed in some of these areas before placing the finished slab concrete. The extent of backfill concrete use is unknown. Concrete placement data indicate that between Sta. 33+00 and 35+00, where the initial chute failure occurred, 2 to 3 times the expected volume of concrete (based on neat lines) was placed. It is not known if any of this additional concrete was placed as backfill concrete. The results of post-failure investigations (Appendix D), including photographic evidence (see Figures A-27 and A-28) indicate that the depth of overexcavation and resulting thickness of concrete were probably not uniform.

**Figure A-27: Damaged area downstream from Sta. 33+00. Note the concrete slab thickness varies in the center of the chute (as discussed in Appendix D).**

**Figure A-28: Damaged area downstream from Sta. 33+00. Note the concrete wall base thickness varies. Also note the unevenness of the broken rock in the foreground.**

Generally, the rock foundation would have high points and low points in the excavation, based on how the rock foundation broke during excavation. This would produce variable thickness in the chute slab. Figures A-27 and A-28 show varying thickness in the chute slab and wall base, respectively. However, in an area just upstream from Sta. 33+00 the foundation seems to be surprisingly uniform for a rock excavation. This can be seen on the underside of the slab in Figure A-19. This seems to be due to an excessive amount of soil on the surface prior to concrete placement, as seen in Figure A-21, as compares to the scoured surface in Figures A-27 and A-28.

As described above, herringbone drains were placed within the chute concrete. The tops of the VCP drain pipes were generally 7 inches or more below the top of the slab (less at the bell joints). In areas where the excavation of the foundation closely matched the design grade, the concrete thickness above the drain would have been half the thickness of the surrounding concrete. This reduced thickness of concrete, spaced longitudinally at 20 to 25 feet, would make an ideal crack inducer for the concrete slab placements that were otherwise 100 feet long between formed contraction joints, as discussed further below. Cracking above the drains was noticed almost immediately after construction (See Appendix F1).

The concrete was placed with a single layer of reinforcement about 3 inches from the top of the slab. While this reinforcement was most likely included to prevent temperature and shrinkage cracking, the single layer was not enough to prevent cracking over the embedded drains (see Figure A-29). Reinforcement crossing over the herringbone drains was often found to have failed in tension at the crack location (see Figure A-30). These issues are discussed further in Appendix D.


**Figure A-29: Excavated section of concrete at herringbone drain location. Note the cracking associated with the drain and the delamination of concrete at the reinforcement layer. (IFT Photo taken during forensic investigation of the chute)**

**Figure A-30: Broken rebar at location of a crack above a herringbone drain. Note that the pipe shown in this photograph was used to support the wooden form for the gravel that encapsulates the drain pipe. (Photo taken during the 2009 repairs)**

## 3.4 Chute Joints

Longitudinal joints were placed between the individual placement lanes. Transverse joints were constructed on 50-foot centers that matched joints in the chute walls. Based on the as-built drawings the 100-foot station joints (Sta. XX+00) were contraction joints that were treated with bond breaker and had dowels crossing the joints, as specified. Intermediate joints (Sta. XX+50) were formed by sawcutting or scoring the concrete on the surface after the slipforming was completed (see Figure A-11). These joints are believed to have continuous reinforcement since there is no break in the reinforcement seen in the construction photos. Observations made following the 2017 spillway chute failure (Appendix D), indicated that these intermediate joints often failed to completely form because the concrete slab was more likely to crack at the reduced thickness sections where the herringbone drains were placed, rather than at the scored section for the intermediate joints.

Three different longitudinal contraction joint details were specified (see the background information related to the spillway design in this Appendix for details). However, field investigations following the chute failure indicate that these joints were not always formed as specified (see Appendix D). Joints between the wall base and the chute slab were designed as lap joints, in which the top of portion of the chute slab rests on the bottom portion of the wall base. The centerline joint was similar. Intermediate lane joints were keyed such that the adjacent slabs could not move independently without shearing the key. Formed transverse joints were lap joints, in which the upper half of the upstream slab overlies the lower half of the downstream slab, such that the downstream slab cannot be lifted without shearing the upstream overlap or “key.”

## 3.5 IFT Findings Related to Construction

The IFT noted several issues related to the construction of the spillways that may have contributed to the problems experienced during the 2017 flooding. Appendix C and D also addresses how these construction related issues. Construction issues are discussed here:

- The foundation conditions achieved during construction were not as expected based on the design and specifications. In places where overexcavation was necessary based on the specifictions requirements, DWR did not direct the contractor to perform the necessary overexcavation and placement of backfill concrete. The IFT believes, that this led to conditions in the foundation where the required cleanup could not be achieved. The specifications requirements were relaxed, and as a result, the foundation was either unbonded or weakly bonded to the concrete chute slab.
- Foundation anchor embedments were not designed to develop the full strength of the No. 11 bars. However, where the foundation cleanup was poor, especially in the area of the initial chute failure, it may have been difficult to determine in the field if the foundation conditions matched thos in the area of anchor testing. The IFT believes that in some areas, cinsiderab increase in embedment length into the foundation would have been needed to achieve the reqired pull-out strength.
- The drain sizes were changed after the bid specifications were completed. It appears that pipe sizes were increased without any other recommended design changes to the cute slab. The increased size of the herringbone pipes made cracking of the slab above the pipes more likely.
- It is apparent that DWR staff expected rock points surrounded by “compacted clayey material” to provide resistance to sliding of the chute slabs. However, it appears that large sections of slab possibly whole placements) had very few exposed rock points that were firmly anchored to the foundation.
- Most chute joints were intended to be unbonded contraction joints with slip dowels passing through the joint. The intermediate joints at 50-foot stationing (Station XX+50) were treated with a sawcut or by scoring the surface to produce a crack inducer, but were otherwise not constructed as contraction joints. This detail made it more likely for cracking to occur above herringbone drain pipes, than at these “joints.”

## 4.0 SPILLWAY CHUTE CONSTRUCTION TIMELINE

Information included in this portion of the appendix was taken from the Final Construction Report (FCR) [A-4]; the Final Geologic Report (FGR) [A-5]; and DSOD Inspection Reports (DSOD), Engineer's Daily Reports (EDR), and Construction Progress Reports (CPR) from the period of spillway construction. The FCR and FGR were provided to the IFT as formal reports, the other documentation was produced regularly during construction, and compiled by DWR as PDF files. They were not part of any known formal report. The EDRs were hand-written, and parts were difficult to read, so they are often not quoted word-for-word by the IFT, but were instead paraphrased. The DSOD Inspection Reports and CPRs were in Memorandum format, dated as shown to the left of the summaries below.

The IFT notes that the inventories of available DSOD Inspection Reports and Construction Progress Reports are not believed to be complete. It appears that some reports are missing from the inventory. The timeline below is based on the available reports, and is more detailed for the chute slab sections in the area of the initial slab failure on February 7, 2017, specifically the chute slab sections between Sta. 33+00 and 36+00. However, key discussions for work in other areas of the spillway was captured where information was available to the IFT, to give a sense of how things developed during construction.

As discussed elsewhere in this appendix, the IFT was given a relatively complete set of as-built drawings and a few key bid specifications drawings for the spillway. There were significant design changes that took place between the bid drawings and the as-built drawings. However, there was no documentation available to explain all the as-built changes, or when they were implemented. The IFT assumes as-built changes applied throughout the construction of the spillway chute. It is also important to note that the reference materials discuss what appears to be changes from the original specifications paragraph requirements, but no documentation could be found for these changes.

01/12/1965 DSOD Memorandum of Office Review [A-12] states the following:

"The specifications provide for excavating to rock which is no less competent than moderately hard or moderately weathered rock. Therefore, backfill concrete will probably be required for the purpose of filling deeply weathered pockets."

"Change detail for placing V.C.P. The detail shown on the plans shows V.C.P. to be laid on rock, and porous backfill concrete to be placed around… Provide details on plans, showing foundation drainage pipe where drain lines cross over backfill concrete…Suggest another longitudinal collector drain be placed down the spillway chute…Show grading of gravel around spillway backfill drain…the designers said they planned to modify the drainage."

"The stability of the flood control structure and the gravity overflow structure is considered to be adequate. The assumption of full hydrostatic uplift at the heel and acting over the total base of the dam during the P.M.F. is considered to be quite sever. The small amount of tension in the heel under these conditions is considered to be acceptable." (Note that the IFT expressed concern with these calculations as indicated elsewhere in this appendix.)

"Providing no unsafe condition is discovered in the review of the proposed changes and the completed plans, structure is considered be safe."

01/17/1966 Memorandum indicating revisions to the chute drainage system design details on Drawing A-3B9-1 [A-13].

02/24/1965 DSOD Memorandum of Office Review [A-14] states the following:

Following are changes and provisions requested and comments thereon:

"Change detail for placing V.C.P. The details on plans showed the V.C.P. to be laid on rock and porous backfill concrete to be placed around it. The backfill concrete was changed to a select gravel. The gravel backfill is graded between 1½ inches and ½ inch. A polyethylene film is to be placed over the gravel backfill."

"Provide details on plans showing spillway slab foundation drainage where drain lines cross over backfill concrete. The detail provides for 3-inch weep holes…"

"Suggested another longitudinal collector drain be placed down the spillway chute. This was not done, but its inclusion is not essential to safety."

"Instability of the gravity overflow structure. This structure was revised further and is still unsatisfactory."

"It is considered that the changes requested by this office have been satisfactorily accomplished in regards to safety of the structure."

03/08/1966 Memorandum indicating revisions to the chute wall details on Drawings A-3B9-2, A-3B-3, and new Drawing A-3B9-7 [A-15].

05/26/1965 Bid Opening for Oroville Spillway (FCR)

06/02/1965 Contract No. 345284 awarded to Oro Pacific Contractors and George FarnsworthConstruction Corporation (FCR)

06/25/1965 Notice to Proceed (FCR)

08/11/1965 Start of spillway excavation (FCR)

01/25/1966 Start of Emergency Spillway concrete (FCR)

03/29/1966 DSOD Inspection by J.F. Chaimson:

"…100-foot long section of chute foundation approximately to grade, at Sta.32+00. I estimated the following:

a. About ¼ of the area was in hard rock knobs up to 2½ feet above grade.
b. About ¼ of the area was weathered and highly weathered material associated with fairly broad shears.
c. The remaining ½ of the area would show moderately weathered rock when cleaned up.

In response to questioning, I indicated that this would not be acceptable as typical foundation for the structure as presently designed. I did agree that, except for the highly weathered material in the shears, the weathered rock would be adequate to hold keys in, particularly on the fairly mild slope.

Apparently, it is planned to design keys for the weathered areas. Mr. Tunstall will submit such a design to our Design Review Branch." (Note that the IFT did not find any indication that these "keys," which are thought to be foundation keys, were ever designed.)

04/19/1966 The IFT had access to the agenda prepared for the April 19-20, 1966 Consulting Board meeting. A photo showing the Consulting Board at spillway chute Sta. 31+50 was included with the DSOD reports (Figure 31). (No Consulting Board report of this visit could be found by DWR).

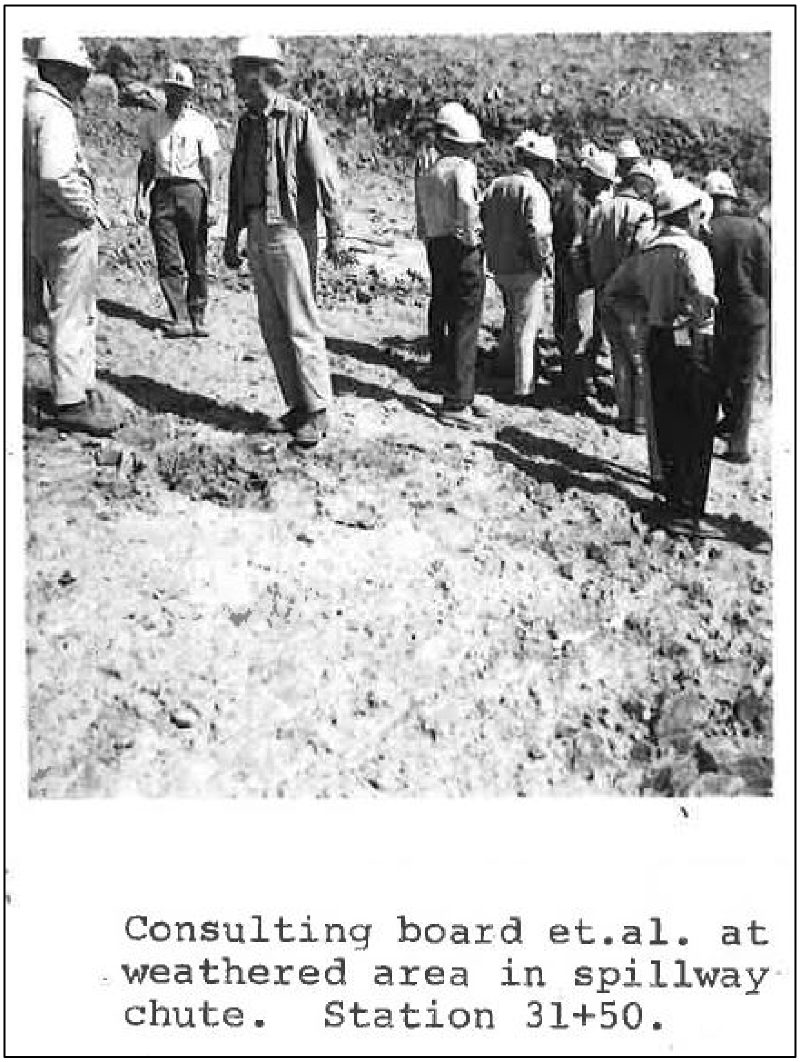


**Figure A-31: Photo from the February 12, 1966 DSOD Inspection Report**

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|---|---|
| 04/21/1966 | Memorandum discussing Oroville Dam Consultant Board meeting of April 19-20, 1966 [A-16]. Mr. Chaimson writes:<br>"Spillway<br>An additional question was presented to the board regarding foundation treatment in the weathered rock area around Station 31+00. This was described in my memorandum of inspection dated March 29, 1966." (Note that DWR was unable to locate the actual Consulting Board report for this meeting.) |
| 05/10/1966 | Start of concrete in Flood Control Outlet (Service Spillway) (FCR) |
| 07/28/1966 | Memorandum indicating revisions to the chute drainage system and terminal structure design details on Drawing A-3B9-1 and A-3B9-4 [A-17]. |
| 08/01/1966 | Drawing A-3B9-1 has revised drainage system details and Drawing A-3B9-4 has dowels added and a revised drainage system. (As-Built Drawings) |
| 08/11/1966 | Completion of Emergency Spillway (FCR) |
| 09/08/1966 | Start of concrete in the spillway chute (FCR) |
| 09/14/1966 | DSOD approved chute foundation right of centerline from Sta. 26+00 to 31+00<br>"Rough excavation in the area approved above indicates good rock knobs will be exposed over 80 percent of the total area." |
| 09/27/1966 | DSOD approves all chute floor slab foundations from Sta. 26+00 to 31+00, and Sta. 26+00 to 27+00 left of centerline.<br>"Floor slab foundation approved like the one just right of centerline was in an area of shears and more weathered rock. An estimate of 50 percent of the area was hard rock points capable of bonding to the slab. The areas below and to the left are much harder rock and will support more than their share." |
| 10/01/1966 | Excavation - Cleanup with heavy equipment between Sta. 29+00 and 34+00, included blasting on left side near Sta. 34+00 (EDR) |
| 10/04/1966 | Washed down Lanes 1 and 2, Sta. 29+00 to 30+00, Jackhammer crew worked on high points in Lane 5, Sta. 26+00 to 27+00. (EDR) |
| 10/05/1966 | Washed down Lane 5, Sta. 27+00 to 30+00. Drill hole depths were discussed with the Superintendent because they were not 5 feet deep after wash-down. The Supervisor ordered the Drill Boss to drill holes 6 feet deep whenever they were not in bare rock. (EDR) |
| 10/08/1966 | Excavation – Cleanup with heavy equipment between Sta. 30+00 and 33+00 (EDR) |
| 10/10/1966 | Chute placement in Lane 3, Sta. 29+00 to 30+00. Finishers did a poor job on the expansion joint groove at Sta. 29+50. (EDR) |
| 10/13/1966 | Lane 1 (right wall) placement, Sta. 32+00 to 33+00. Shot rock in Lane 3, Sta. 33+70 to 37+50, produced very good riprap material. (EDR) |

10/14/1966 Small slide occurred overnight on the right side slope, Sta. 33+00 to 33+50. Sta. 32+00 to 33+00 needed to be cleaned and the drains reset. The contractor insisted on air cleaning, which was not satisfactory. (EDR)

10/17/1966 Chute floor slab placement Sta. 31+00± (DSOD)

"Rough excavation in the spillway was exposed on right side Sta. 31+00 to 32+00. Hard rock was exposed on a very rough surface. The weathered rock anticipated in this area is apparently a little lower."

Contractor started three shifts for placement of the spillway chute. (EDR)

Also placed concrete in Lane 4, Sta. 29+00 to 30+00. The contractor was told air only cleanup was leaving unsatisfactory results. Contractor said they would blow it off then wash it down. Lane 5, Sta. 28+00 to 30+00, and Lane 3, Sta. 30+00 to 31+00, were cleaned and drains placed in Lane 5. Anchors drilled in Lane 2, Sta. 30+00 to 32+00. A rock slide occurred on the right side at Sta. 29+00, breaking the 12-inch riser and 12-inch drain. Final cleanup in Lane 6, Sta. 29+00 to 30+00. (EDR)

10/18/1966 Concrete placements in Lane 6, Sta. 28+00 to 29+00, and Lane 4, Sta. 28+00 to 29+00. Anchor bars placed in Lane 2, Sta. 30+00 to 31+00. The contractor was warned about using the wrong anchor bars and cutting too much off. (EDR)

10/19/1966 Concrete placement Lane 5, Sta. 29+00 to 30+00. The contraction joint groove at Sta. 29+50 was off by 2 inches downstream compared to the Lane 6 joint. According to on inspector, the "[groove] @ 29+50 is a mess." Also, the Lane 5 concrete was ½-inch higher than Lane 6 at Sta. 29+50. Carpenters pulled off the top of the bulkhead at Sta. 29+00 too early leaving damage the finishers needed to correct. The contraction joint had a large "chipped" area. (EDR)

10/20/1966 Concrete placement in Lane 3, Sta. 30+00 to 32+00, and Lane 5, Sta. 28+00 to 29+00. (EDR)

10/21/1966 Water cleaned chute foundation Lane 4, Sta. 30+00 to 32+00 and Lane 1, Sta.34+00 to 35+00. (EDR)

10/24/1966 Labor crews worked on cleanup, drains, and anchor bars Sta. 30+00 to 35+00.

"Placed 796 $yd^3$ in chute slab (Sta. 30+00 to 32+00, lane 2) approximately one half of this is within the neat lines." (taken to mean this area was overexcavated beyond the pay lines) (EDR)

10/25/1966 Labor crews worked on cleanup and drains, Sta. 30+00 to 35+00. Concrete placed in Lane 4, Sta. 30+00 to 32+00, and Lane 1, Sta. 34+00 to 35+00. (EDR)

10/26/1966 Labor crews worked on cleanup, drains, and anchor bars Sta. 30+00 to 34+00. Concrete placed in Lane 6, Sta. 30+00 to 32+00. (EDR)

10/27/1966 DSOD approves foundation in Lane 3 from Sta. 32+00 to Sta. 33+00

"The foundation rock in the approved area is about the softest in the whole chute." The cleanup was not good but judged adequate. It was washed several times creating overbreak of 1 to 2 feet. (DSOD)

Placed concrete in Lane 5, Sta. 30+00 to 32+00. Very deep hole in the middle. "Labor crew spent most of the day cleaning out and making corrections in Lane 5, Sta. 35+00 to 36+00." Lane 2, Sta. 32+00 to 33+00 was washed down to remove ½" to 1½" of mud. Cleanup in Lane 2, Sta. 32+00 to 33+00 was found to be unacceptable. Mud covered the entire surface, but the contractor said they could not do any more with it because there was no rock. They were told to clean it more with air and water. (EDR)

10/28/1966 Clean-up of Lane 1, Sta. 35+00 to 36+00 and set drains. Anchor bars installed in Lane 3, Sta. 33+00 to 34+00. They had to cut off and reinstall eight or nine anchors because they were not straight. Pipe installed and backfilled in Lane 3, Sta. 33+00 to 34+00. The cleanup was satisfactory, but very difficult to achieve. Concrete placed in Lane 6, Sta. 32+00 to 33+00, and Lane 3, Sta. 32+00 to 33+00. (EDR)

10/29/1966 Cleanup and wash down of Lane 5, Sta. 32+00 to 33+00, Lane 4, Sta. 33+00 to 34+00, and Lane 6, Sta. 35+00 to 37+00. Anchor bars set in Lane 3, Sta. 33+00 to 34+00 and Lane 4, Sta. 32+00 to 33+00. (EDR)

10/31/1966 Concrete placement in Lane 2, Sta. 32+00 to 33+00. A hump in the concrete at Sta. 32+12 had to be corrected. Placement in Lane 4, Sta. 32+00 to 33+00, and Lane 1, Sta. 35+00 to 36+00. The second conveyer was too far upstream and concrete could not be placed close to the slipform. The contractor was told the cleanup in Lane 4 was not satisfactory. There was loose clay and shot rock on the surface, with very little good rock. The laborer refused to do more until he was told to do so by his boss. (EDR)

11/01/1966 Placed concrete in Lane 5, Sta. 32+00 to 33+00, and Lane 4, Sta. 32+00 to 33+00. (EDR)

11/02/1966 Placed concrete in Lane 3, Sta. 33+00 to 35+00. The shallow placement was difficult, and a drain blocked the flow of concrete. There were a lot of surface voids Sta. 34+40 to 34+45. The placement got deeper about half way. Concrete was delivered with 6-inch rocks, and one of the rocks broke the drain pipe. There was a delay while the pipe was replaced. Dirty gravel around the drain had to be replaced at Sta. 33+00. Placed concrete in Lane 6, Sta. 34+00 to 35+00. (EDR)

11/03/1966 Placement in Lane 3, Sta. 33+00 to 35+00 continued. Placement started in Lane 2, Sta. 33+00 to 35+00. Laid 6-inch pipe in Lane 6, Sta. 34+00 to 35+00, and Lane 6, Sta. 34+00 to 35+00. Reminded the contractor to clean up the foundation before placing drains, set anchors to grade within ½-inch and keep equipment away, and

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|---|---|
| | cleanup needed to expose 50 percent rock, and most of the fines needed to be removed. (EDR) |
| 11/04/1966 | Placed concrete in Lane 1, Sta. 36+00 to 37+00, and Lane 4, Sta. 33+00 to 35+00. (EDR) |
| 11/04/1966 | Cleanup of Lane 5 with air and water showed that it was mostly well weathered rock and clay seams. Laid 6-inch pipe in Lane 5 starting at Sta. 33+00. (EDR) |
| 11/07/1966 | Rain the day before (Sunday) triggering several slides on the side slopes. Placed concrete in Lane 6, Sta. 35+00 to 36+00, and Lane 5, Sta. 33+00 to 35+00. (EDR) |
| 11/08/1966 | Placed concrete in Lane 6, Sta. 36+00 to 37+00, and Lane 3, Sta. 35+00 to 37+00. (EDR) |
| 11/09/1966 | Check of transverse slip dowels Sta. 35+00 to 37+00 found 55 bars out of position by between 2 and 12 inches. Dowel blockouts filling with grout, preventing them from going in. Cleanup in Lane 4, Sta. 35+00 to 37+00 was not adequate and this area needed to be further cleaned. The contractor wanted to clean as the drains were laid. (EDR) |
| 12/14/1966 | Chute slabs placed from Sta. 26+00 to 41+00 (DSOD summary)<br>"Cleanup of foundation under spillway floor slabs was noted to be very poor considering good rock. Many rock fragments, covering more than 50 percent of the area, were left in place. This brought to Mr. Hand's (DWR Inspection) attention and requested that a better job be done. Probably no unsafe condition is created." |
| 12/31/1966 | DSOD Report of Inspection of Dams (from 12/14/1966 inspection) [A-18].<br>"It was noted that cleanup of the foundation under the floor slabs was not satisfactory since many rock fragments, covering more than 50 percent of the area, were left in place. Mr. Hand was requested to have this unsatisfactory condition corrected." |
| 01/17/1967 | Memorandum indicating revisions to the chute drainage system and other details on Drawings A-3B9-1 and A-3B9-2 [A-19]. |
| 02/09/1967 | Memorandum indicating revisions to the chute drainage system on Drawing A-3B9-1 [A-20]. |
| 02/27/1967 | DSOD Memorandum "Revised Drawings"<br>DSOD expresses concern with potential for lifting chute slabs because surface water drains into collector drains. DSOD recommends an impervious cap on the pervious material surrounding the collector drains. |
| 08/14/1967 | Completion of concrete in the spillway chute (FCR) |
| 08/25/1967 | Completion of concrete in Flood Control Outlet (Service Spillway) (FCR) |

10/26/1967 Change Order 21 was issued (FCR)

Directed the contractor to make changes to the drainage system based on BOC recommendations. This required resloping and scaling of the spillway chute rock slopes affected by the revised drainage system. The change covered compensation for the drainage system and resloping, but did not include adjustment for the completion time.

02/20/1968 Completion of Contract (FCR)

## 5.0 IFT FINDINGS RELATED TO THE TIMELINE

- Documentation from the Consulting Board is lacking to describe their reason for design changes related to the drainage system and their recommendations related to foundation quality. Other documentation discusses the request to increase the size of the drains and the slope (creating the herringbone pattern), but not in the words of the Consulting Board. The Consulting Board was also asked about foundation conditions that did not meet the design intent, but their response could not be located by DWR in their files.
- There is no documentation describing the reason for changes to chute slab joint details changing the joints from expansion joints to contraction joints.
- There is no documentation for the apparent relaxation specifications related to the chute foundation excavation and cleanup. While the Consulting Board was asked about this, the IFT cannot conclude that relaxation of specifications requirements was approved by them.
- From the documentation available to the IFT, it is apparent that even as the foundation was being excavated by heavy equipment that there were areas of the foundation near location of the February 7, 2017 chute slab failure where the foundation was more weathered than anticipated by the designers and geologists. The Consulting board was asked to look at this area (see above), and at some point, it was decided that only 50 percent of the exposed foundation needed to be hard rock, and the remaining portions could be compacted clayey material. However, as the concrete placements approached the area downstream from Sta. 33+00, even the relaxed foundation requirements could not be met without significant overexcavation. High pressure washing was resulting in overexcavation. Although the discussions between DWR and the contractor was no captured in the documentation provided to the IFT, it appears that the contractor believed that the foundation needed to be overexcavated to meet specifications requirements, and the contractor wanted compensation for the overexcavation. While DWR conceded that overexcavation was needed in the area of the shears, they apparently did not agree that other weathered areas required overexcavation. Refer also to Appendix C. It would seem to the IFT that relaxation of the cleanup as discussed was an attempt to prevent significant cost overruns that could have occurred if the foundation had been excavated to non-rippable rock and cleaned of all objectionable material as described in the specifications. The result was a foundation that did not meet the original design intent.

## 6.0 REFERENCES

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[A-13] Memorandum To: Mr. H. C. Carter, From: J. A. Wineland, Subject: Oroville Dam Spillway, Specifications No. 65-09, Contract No. 354284, Revised Contract Drawings, January 17, 1966.

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[A-18] Department of Water Resources, Division of Dam Safety, Report of Inspection of Dams, December 31, 1966.

[A-19] Memorandum To: Mr. H. C. Carter, From: J. A. Wineland, Subject: Oroville Dam Spillway, Contract No. 354284, Specifications No. 65-09, Transmittal of Contract Drawings, January 17, 1967.

[A-20] Memorandum To: Mr. H. C. Carter, From: J. A. Wineland, Subject: Oroville Dam Spillway, Specifications No. 65-09, Contract No. 354284, Transmittal of Revised Drawings, February 9, 1967.

# Appendix B

# Hydraulic Analyses

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## 1.0 INTRODUCTION

The hydraulics of the flow for a variety of discharges were analyzed for the Oroville Dam spillway chute to obtain an understanding of the velocity and cavitation characteristics for historic discharges and those that occurred during spillway chute slab failure on February 7, 2017.

The computer program used in this analysis was originally developed by Dr. Henry T. Falvey in Fortran Code, see Reference [B-1]. That program was subsequently converted for an Excel Spreadsheet by Falvey and an engineer from the US Department of Interior, Bureau of Reclamation. The program outputs the hydraulic and cavitation properties of the flow on steep chutes and spillways for a specified discharge and reservoir elevation. It uses the Darcy-Weisbach friction factor to compute the frictional resistance instead of the Manning equation that was developed for flow on slopes of less than 10-degrees.

The historic reservoir elevations were used to determine the discharges that were simulated. The rating curve for the fully open gates was used for input into the computer program.

## 2.0 HYDRAULIC PROPERTIES

The Hydraulics Excel Spreadsheets display not only the geometry, flow depths, and velocities, they also display the mean air concentration, the boundary layer thickness, and an indication that roll waves may develop. These latter three properties are important in evaluating the amount of air entrainment and bulking that may occur on the chute. Reference [B-1] presents the derivations and equations for the values in each column of the spreadsheets.

The velocity on the chute is the most important factor in evaluating the uplift pressures that could develop because of cracks that form over the herringbone drains and at the contraction joints as well as at open joints, spalls and failures of repairs. The Hydraulic Sheets in the Appendix show that the velocities near the initial chute failure location downstream of Station 33+00, were all close to 100 ft./sec.

Turbulence at the water surface is needed to entrain air. The turbulence is produced by the roughness of the surface boundaries. A certain distance is required for the turbulence from the boundary to reach the water surface. The thickness of the turbulence of the flow along the boundary is called the boundary layer. A boundary layer begins to grow at the gates. The thickness of the boundary layer grows until it reaches the depth of flow. At this point surface aeration of the flow can begin. This is shown in Figure B-1. Once surface aeration has begun, turbulence will propagate the air bubbles deeper into the flow until they reach the invert. This process takes place over a distance that is approximately equal to the distance that was necessary for the boundary layer to reach the water surface. The distribution of the air continues to change downstream until it becomes almost constant. This is called "fully-developed, fully-aerated flow."

Depending upon the invert slope and velocity, an instability can develop on the water surface that create hydraulic bores. The bores are often called "roll waves." The terminology is a bit confusing because some investigators also refer to surges as roll waves. In this document, roll waves refers to small bores that gradually increase in size until they form a large disturbance known as a surge. Surges are common for shallow flows, as shown in Figure B-2. However, they are not often seen with deep flows because long distances are necessary to transition from roll waves to surges. These large surges are also called "slug flow" because the shape of the surge looks something like a gastropod when it moves along the ground.

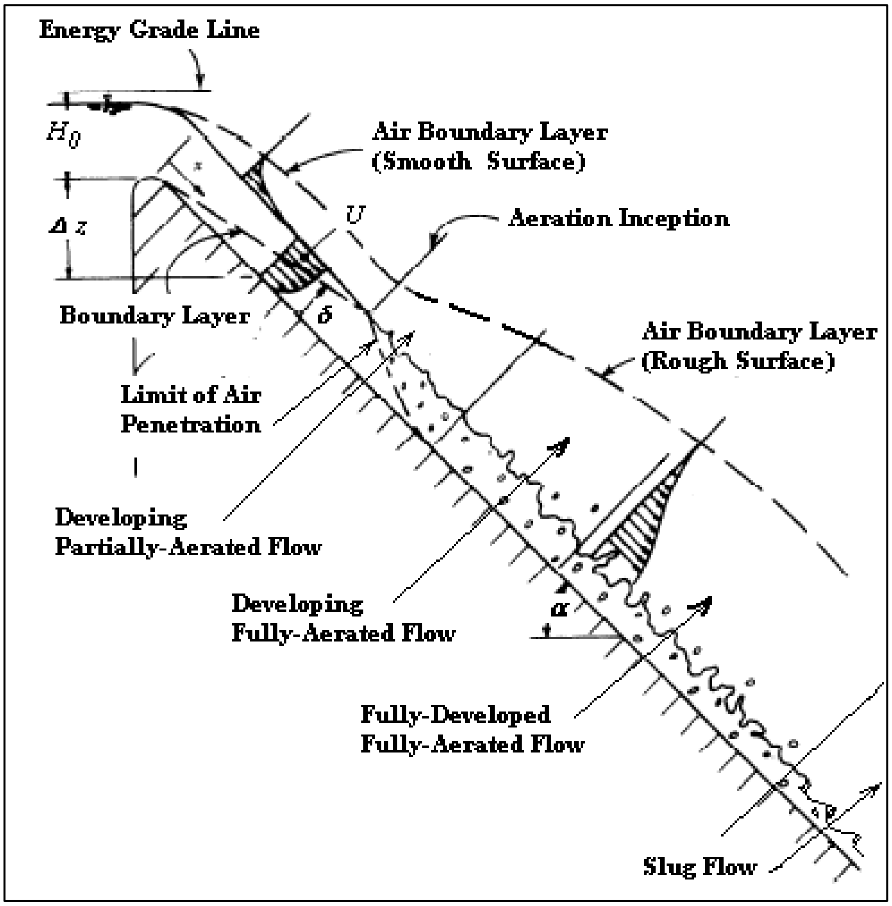


**Figure B-1: Development of Aeration and Slug Flow or Roll Waves in Chutes (Modification of Figure in Reference [B-1])**

Roll waves with larger flows on the Oroville spillway chute are shown in Figure B-4. The distance between the waves is generally random. Each crest entrains air. The randomness of the waves will induce fluctuation pressures on the invert.

**Figure B-2: Slug Flow in Spillway Chute. The circled area shows a flow disturbance at Station 33+00 for very low discharge.**
**Reference [B-4]**

The onset of hydraulic bores or roll waves is predicted by two dimensionless parameters, the Vedernikov and the Montuori numbers, see Figure B-3. The Vedernikov number is a parameter that indicates the stability of the flow. The Vedernikov number, V, is defined as:

$$V = 2F\left(1 - R_h \frac{\partial P}{\partial A}\right) \qquad (1)$$

In which

A = Cross sectional area
F = Froude Number
P = Hydraulic Perimeter
$R_h$ = Hydraulic Radius

If the Vedernikov number is greater than unity, the flow is unstable.

The Montuori number is the ratio between the inertial and friction forces. The Montuori number is given by:

$$M = g\frac{LS}{U^2} \quad (2)$$

In which

g = Acceleration of gravity
L = Length of chute to station
S = Invert slope
U = Mean velocity in chute

These numbers are computed in the computer program mentioned above and they predict when roll waves are possible. The Hydraulic Sheets attached in the Appendix show the stations for which roll waves can develop for various discharges. Roll waves were probably present at Station 33+00 when the initial chute failure occurred.

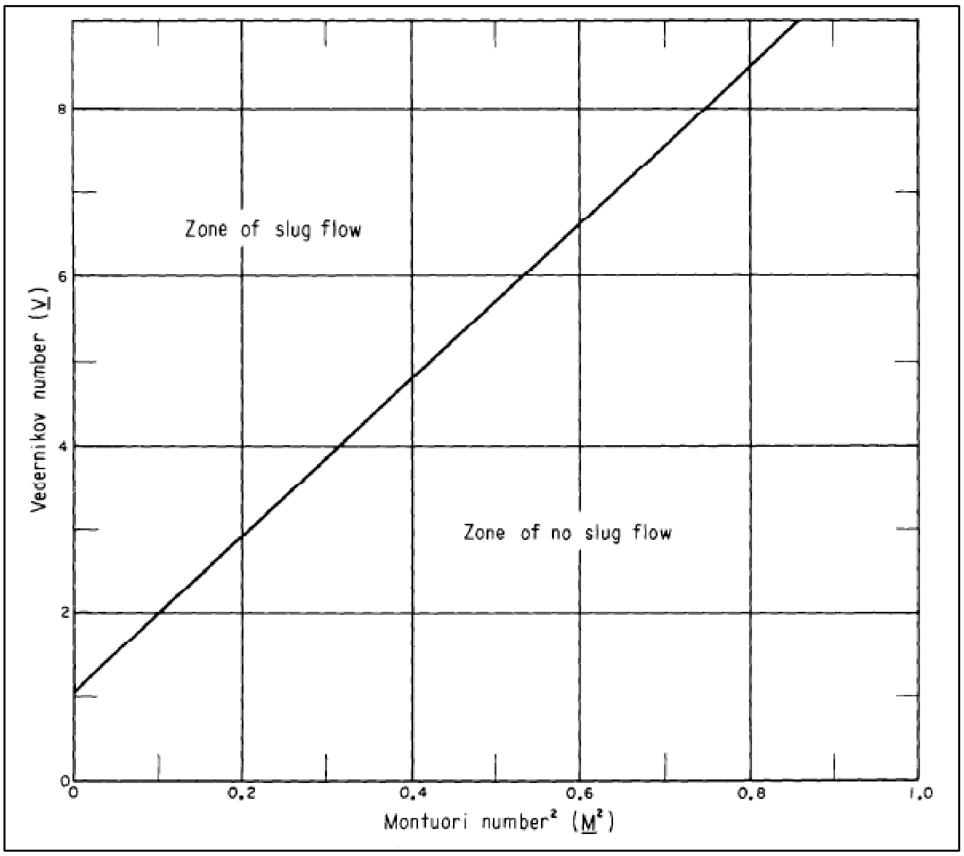


**Figure B-3: Onset of Roll Waves,**
**Reference [B-6]**

Bores or roll waves can be observed on the water surface of the flow for a discharge of about 19,000 cfs in Figure B-4.


**Figure B-4: Roll Waves for 19,000 cfs Discharge, Reference [B-5]**

The air entrainment characteristics of the chute are evident from a satellite photo as shown in Figure B-5. The gates entrain a lot of air, but the effect of the turbulence generated by the invert has not reached the water surface. As a result, the air rises to the water surface and is released to the atmosphere. This is called "detrainment." The rapid detrainment of air immediately downstream of the gate structure is evident on the upstream end of the chute. This indicates that aerators in this region would probably not be effective. Further downstream, the point at which the boundary layer reaches the water surface is evident. The exact location is not well defined because the turbulence will cause the air to be entrained randomly near the water surface. Further downstream, one can see well aerated flow and the development of roll waves that enhance the air entrainment.


**Figure B-5: Satellite Photo of Spillway**
**Modified from Google Earth**

## 3.0 CAVITATION PROPERTIES

Cavitation is a process like boiling. Water is changed from a liquid state into water vapor. With boiling, the process is caused by the addition of heat. With cavitation the change in state is caused by decreases in pressure. If the water pressure decreases below vapor pressure, the liquid water will change into bubbles of water vapor. When high velocity flowing water passes over a surface irregularity, the pressure near the irregularity can decrease rapidly leading to the formation of cavitation bubbles near the flow surface. As the bubbles pass downstream into a higher-pressure region, the bubbles collapse as the vapor changes into liquid water again. The collapse of the bubbles creates high pressure waves. With time, these pressure waves can damage any surfaces from concrete to stainless steel. The time needed to damage the surface depends upon the strength of the material. The process of cavitation is described by a dimensionless number that relates the flow velocity and the pressure. The number is known as the cavitation index and is defined as:

$$\sigma = \frac{\frac{P}{\gamma} - \frac{P_v}{\gamma}}{\frac{V^2}{2g}} \qquad (3)$$

In which

g = Acceleration of gravity
P = Ambient Pressure at the flow disturbance

$P_v$ = Vapor Pressure of the water
ϒ = Specific Force of Water
σ = Cavitation Index

An examination of Equation (3) shows that the cavitation index is the ratio of the potential energy of the flow relative to vapor pressure(numerator) over the kinetic energy (denominator). Thus, as the kinetic energy increases for a given flow depth, the ratio decreases and cavitation is more likely to develop.

Three requirements are necessary for damage to develop on a chute due to cavitation: 1) the cavitation index of the flow must be lower than a critical value, 2) a source must be present to initiate cavitation, and 3) sufficient time must elapse to damage the surface. Falvey [B-1] examined the cavitation experience on several dams worldwide and found that these parameters can be shown graphically, as illustrated in Figure B-6. Note that all damage on the spillways and chutes began with a cavitation index (σ) of the flow less than 0.2.

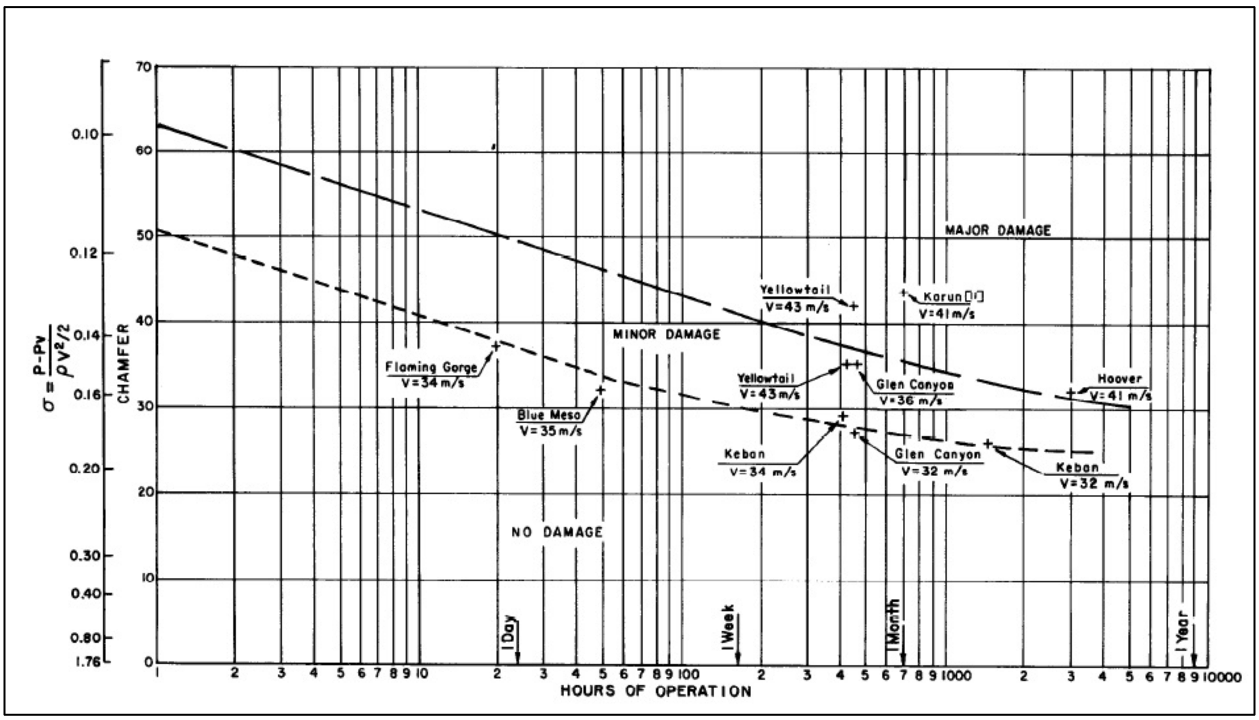


**Figure B-6: Damage Experience in Spillways and Chutes, Reference [B-1]**

The data points can be expressed qualitatively though a parameter known as the Damage Index. The Damage Index is made up of two factors; a Damage Potential and time of operation at a given discharge. The Damage Potential is a function of the cavitation index of the flow, the cavitation index for the initiation of damage, and the relative velocity. The Damage Potential is a non-dimensional number that describes the aggressiveness of the cavitation and the susceptibility of a certain offset size and shape to cavitation damage. The Damage Index is a quasi-qualitative measure of the severity of the cavitation damage as a function of discharge and time. Details of these parameters can be found in reference [B-1].

The relationship between the Damage Potential and the Damage Index is given by Equation 4:

$$D_i = D_p ln\left(\frac{t}{t_o}\right) \tag{4}$$

In which

$D_p$ = Damage Potential
t = Elapsed Time
$t_o$ = Time at Start of Flow

The critical values of the Damage Potential and Damage Index are given in Table 1.

**Table 1: Design Values of Damage Potential and Damage Index**

| Damage | Damage Potential[1] | Damage Index |
|---|---|---|
| Incipient | 500 | 5,000 |
| Major | 1,000 | 10,000 |
| Catastrophic | 2,000 | 20,000 |

[1]The values of the Damage Potential were obtained by solving Equation 4 for Damage Potential with $t/t_o$ = 22,000 hours or 2 ½ years. A shorter time interval will produce larger values of the Damage Potential.

The Cavitation Properties Sheets in the Appendix attachments show that cavitation damage is likely for extended operational times below station 31+00 with discharges equal to or greater than 100,000 cfs. Therefore, an aerator would be appropriate at this location to protect the rest of the chute downstream of this station. No aerators are needed upstream of station 31+00. Note that this criterion matches very well with the observed minimum cavitation index of the flow that is equal to 0.2. That is, cavitation damage will not develop anywhere on the chute with a cavitation index of the flow greater than 0.2.

The lack of observed cavitation damage to the chute below station 31+00 can be attributed to the relatively short operating times at discharges greater than 100,000 cfs in the history of the project; the formation of roll waves at discharges less than 100,000 cfs; and the lack of large offsets on the chute. Note that the damage index exceeds 5,000 (Incipient damage) only for offsets greater than 1/2-inch for all discharges and offsets greater than 1-inch for discharges greater than 100,000 cfs. It does not predict major damage for any flow rate.

Cavitation damage inception would not be expected anywhere on the chute for discharges less than about 50,000 cfs. Similarly, cavitation damage inception would not be expected anywhere on the chute above station 31+00 for any discharge. No evidence of cavitation damage has been observed in any of the inspection reports around this station since the construction of the spillway.

The time history of the failure in the spillway chute is shown in Figure B-7. The flow was ramping up from a discharge of about 42,000 cfs when the failure occurred. At the discharge of 54,500 cfs, cavitation would not occur around Station 33+00. The flow was shut off around 11:30 AM and was essentially stopped around 12:20 PM.

Drone inspections of the sections of the chute remaining in place downstream of the chute failure area did not indicate evidence of cavitation damage. In addition, a careful onsite examination of the remaining lower chute sections by a member of the Independent Forensics Team (IFT) could not find any evidence of cavitation damage at supposed locations of damage. Therefore, we can conclude that cavitation was not a contributor to the failure that was observed on February 7th, 2017.

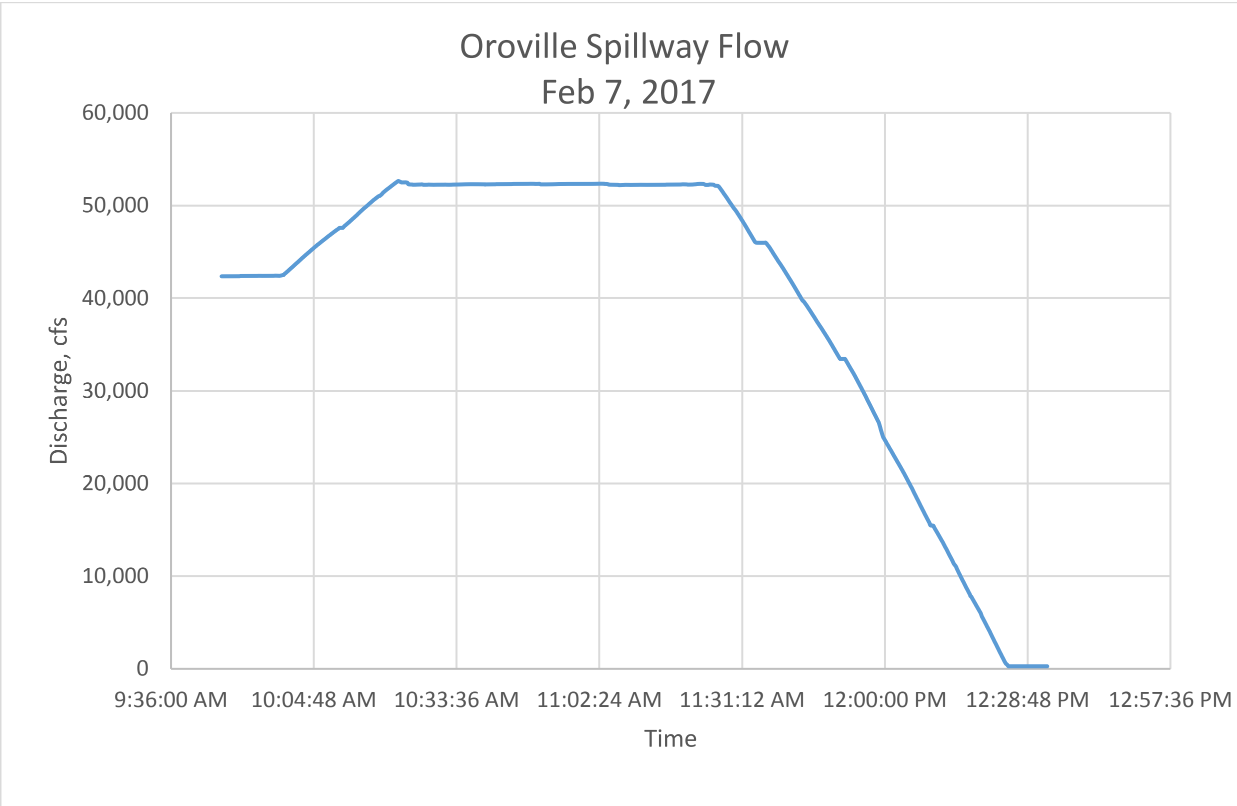


**Figure B-7: Time History at Time of the Chute Failure**

The only observable damage in any of the inspection reports was reported at the deflector blocks (chute blocks or dentates), by the DWR Board of Consultants, on November 18, 2015, as shown in Figure B-8. They are standing at the dentates for the chute. Note the concrete patch to the right of the Board members. Patching of the dentates is sometimes required after significant releases. The only problem with the conclusion of the Board of Experts is that the patch is not in a location where cavitation would occur. It is however, in a location that is susceptible to impact damage.

**Figure B-8: Observation of Cavitation Damage to Dentates, Reference [B-7]**

## 3.1 Conclusions

1. Cavitation was not a contributor to the failure of the service spillway chute.
2. Roll waves have been very effective in protecting the downstream chute from cavitation damage for the historic discharges.
3. Cavitation damage is predicted for the chute below Station 38+00 with discharges of more than 100,000 cfs. However, the historic operating times with high discharges have been too short, and the surface was too smooth to produce any observable damage, except on the dentates.
4. An aerator is needed at Station 33+00 to protect the chute downstream of Station 33+00 for long term operation with discharges higher than 100,000 cfs.

# 4.0 STAGNATION PRESSURES

## 4.1 Introduction

When a flow of water strikes a vertical plate, part of the flow will be deflected upward, and part will be deflected downward. The point on the plate where the two flows separate is called the stagnation point because at that location the flow velocity on the boundary is equal to zero. At that location all the kinetic energy of the flow is converted into potential energy. The pressure at that location is called the stagnation pressure and it is the highest pressure in all the flow. In hydraulic structures, the vertical plate could be a slab that is raised into the flow or the downstream end of a spall or repair pop-out. At these locations, the stagnation pressure can be transmitted under the slab to create uplift forces on the slab.

Uplift forces on slabs in hydraulic structures have been a topic of interest for safe and reliable operation for many years. Most of the research has focused on turbulent fluctuations under hydraulic jumps in stilling basins. However, in many instances, a chute slab has failed leading to catastrophic damage, as shown in Figure B-9.

**Figure B-9: Damage from Uplift of Chute Slab, USBR Report DSO-07-07**

The transmission of pressures beneath chute slabs into the foundation depends largely on the locations, geometry, and design of the slab joints. If the slabs are not provided with waterstops at the joints, water can leak into the joints.

Some slabs have been designed with the drains located within the slabs, as is the case at for the Oroville Dam service spillway. This design feature produces structural weak spots in the slabs that can lead to cracks over the drains, as shown on Figure B-10. These cracks also allow the penetration of water into the foundation below the slab.


**Figure B-10: Crack over Herringbone Drain at Oroville Dam Service Spillway, IFT Photo**

The flow into joints and cracks as well as the uplift pressures have been studied by Johnson [B-2] and Frizell [B-3] at the US Bureau of Reclamation.

Two conditions need to be analyzed. The first is for no flow through the joint or crack. The purpose of this analysis is to determine how much additional pressure will be transmitted under the slab when the upstream end of the slab is lifted into the flow by a given amount. This pressure can also develop at the downstream edge of a repair patch that has popped out of the flow surface.

This analysis assumes that the foundation material is saturated. The second condition assumes that the foundation material is not saturated and the pressure beneath the slab is equal to atmospheric pressure. The pressure at the entrance to the joint or crack will be equal to the piezometric pressure on the chute. The purpose of this analysis is to estimate the quantity of flow into the slab through the joint or crack.

## 4.2 Analysis

The velocity at any depth as a function of the mean velocity and friction factor for a wide channel at normal depth is given by[1]:

$$\frac{v-V}{V\sqrt{f}} = 2log\left(\frac{y}{y_o}\right) + 0.88 \qquad (5)$$

In which

f = Darcy – Weisbach Friction Factor
v = Velocity at depth equal to
y = Depth in flow
$y_o$ = Depth where the Velocity is Equal to Zero,
Usually equal to the Boundary Roughness
V = Mean Flow Velocity

[1] Rouse, H, 1945, *Elementary Mechanics of Fluids.*

Equation (5) is based on theoretical considerations for turbulent flow and has been validated by physical measurements. It is only valid for flow depths greater than the boundary roughness.

The velocity distribution at Station 33+00 for discharges of 30,000 cfs and 54,000 cfs are shown in Figure B-11.

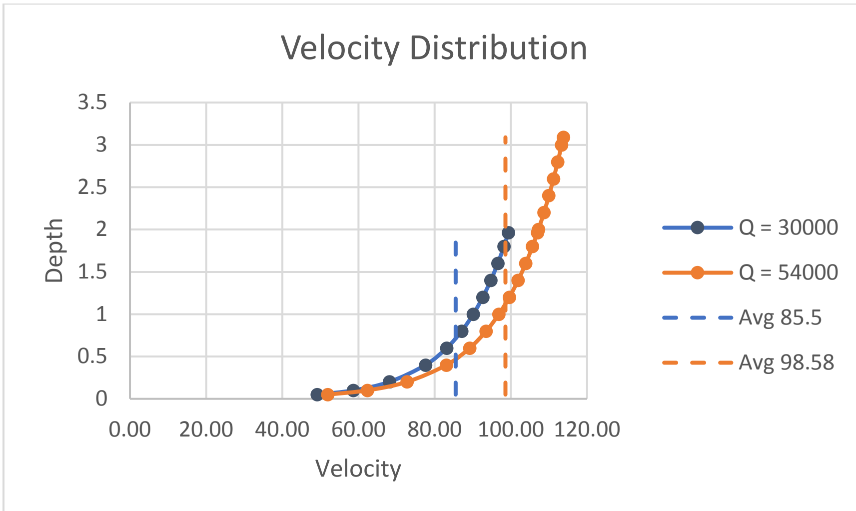


**Figure B-11: Velocity Distribution at Station 33+00**

The stagnation pressure is the pressure on a vertical surface where the approach velocity goes to zero. It is equal to:

$$\frac{P_s}{\gamma} = \frac{v^2}{2g} \tag{6}$$

In which

Ps = Stagnation Pressure
v = Flow Velocity at Offset
g = Acceleration of Gravity
ϒ = Specific Weight of Water

A two-dimensional particle imagery in Reference B-3 by Frizell shows that the stagnation pressure occurs at approximately ½ of the offset height as shown in Figure B-12. The blue vectors in the figure represent the flow velocity. The green vectors are the flow velocity at locations where the data were incomplete. The space covered with dots represents the concrete slabs. The stagnation point can be seen about half way up the vertical face of the downstream slab. Above the stagnation point, all the flow is deflected over the downstream slab. Below the stagnation point, the flow is in a recirculation zone. The recirculation patterns even develop within the gap between the slabs.

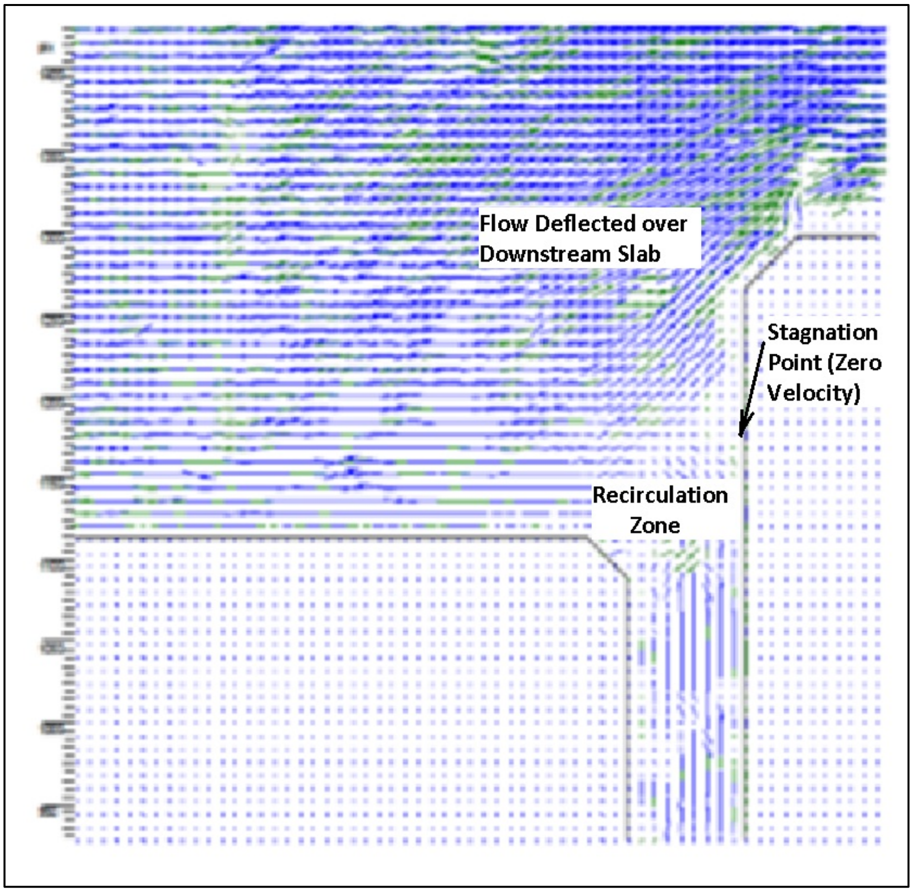


**Figure B-12: 1/8-inch Gap with a 1/2-inch Offset and a Vented Cavity, Reference [B-3]**

The computational fluid dynamics (CFD) studies in Reference B-3 by Frizell show that when the cracks are open to the atmosphere the stagnation pressure is concentrated to a small area around the impact point as shown in Figure B-13a. Note that the stagnation pressure will be equivalent to the velocities on the stream line that approaches the downstream slab. The velocity distribution can be seen by the location where the velocity vectors are shown to begin and end on each side of the figure.

The pressures on the figures are in pounds per square foot, the velocities are in feet per second, and the dimensions are in feet. The velocity distribution used in the numerical analysis is given by:

$$u = 2.5\, V\, ln \left[\frac{y}{y_o}\right] \tag{7}$$

In which

$u$ = Velocity
$y$ = Flow Depth
$y_o$ = Elevation for which velocity = 0
$V$ = Mean Flow Velocity

This equation as given in this reference is not correct because the theoretical development of the equation shows that the term $\sqrt{\frac{f}{8}}$ is missing from the right-hand side of the equation. With this addition and some manipulation of the terms, the revised equation can be shown to be equivalent to Equation (6).

The boundary layer thickness in these simulations was with only a 12-inch flow depth, so the CFD studies simulated conditions very near the beginning of the boundary layer in the field.

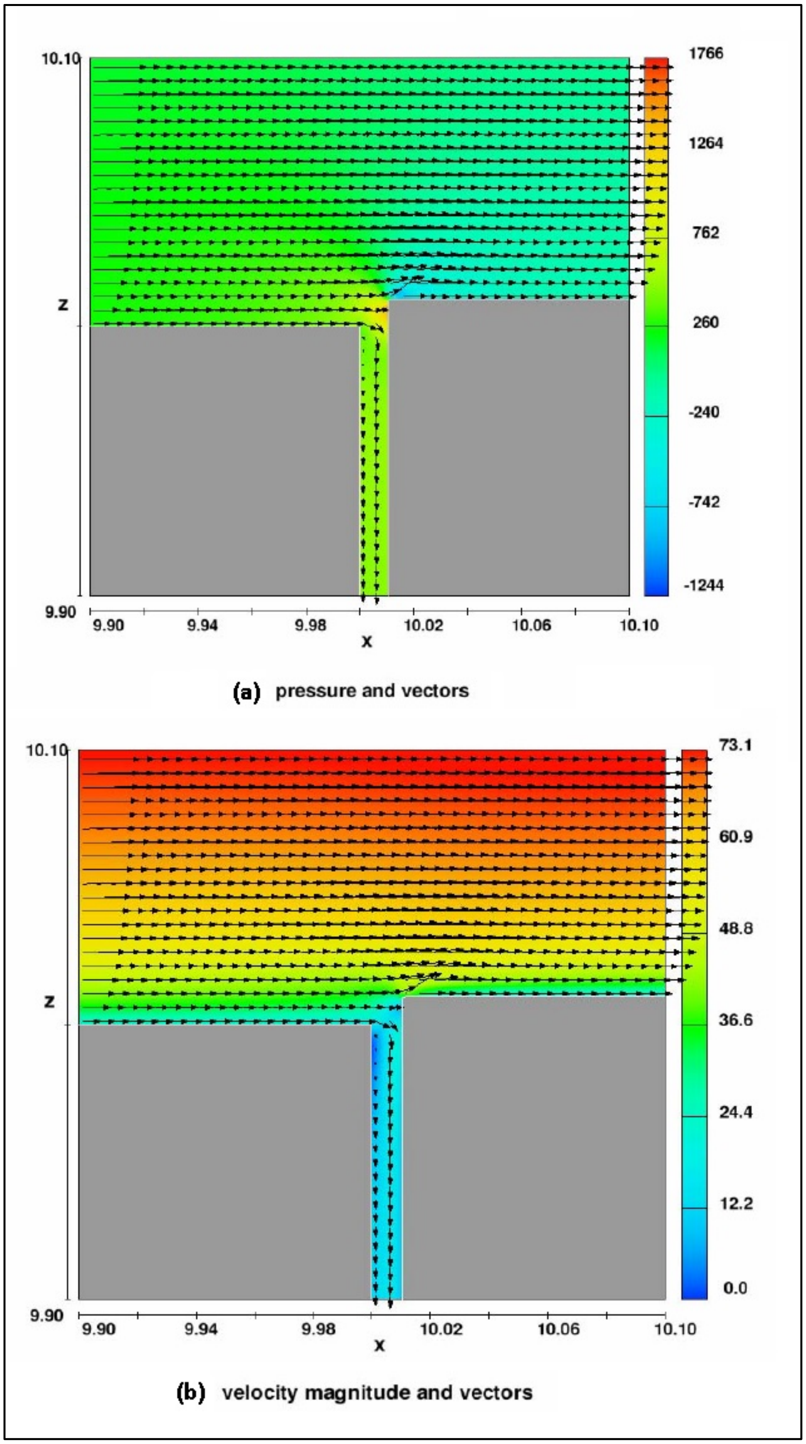


**Figure B-13: Free Flow 1/8th Inch Offset 1/8th Inch Gap, 12-inch Flow Depth, 90-ft/sec Velocity, Reference [B-3]**

However, with a sealed cavity that would simulate conditions with a saturated foundation under the slab, the pressure in the slot is only slightly less than the stagnation pressure as shown in Figure B-14(a).

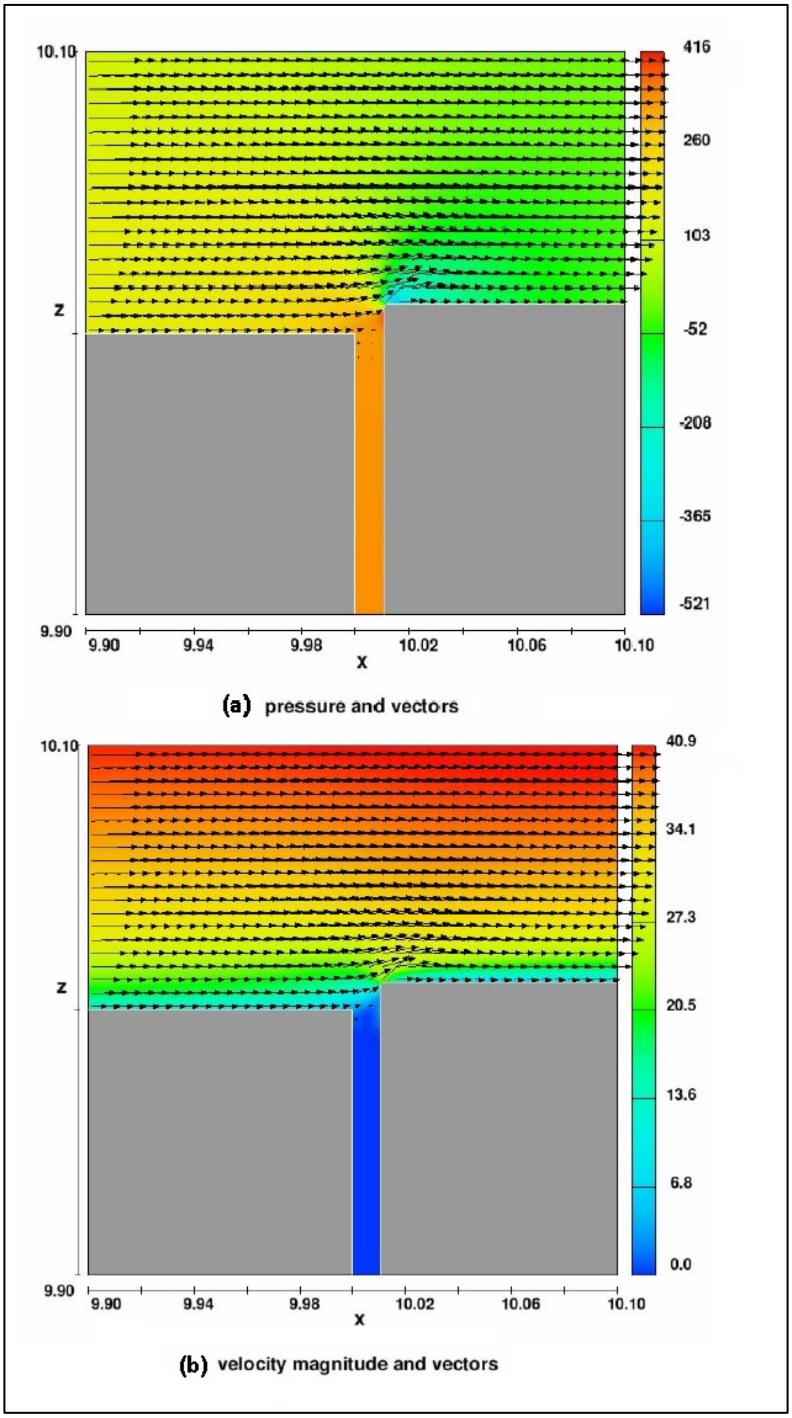


**Figure B-14: Sealed Cavity 1/8th Inch Offset 1/8th Inch Gap, 12-inch Flow Depth, 50-ft/sec Velocity, Reference [B-3]**

Equations 5 and 6 can be used to estimate the stagnation pressure using the depth of flow and average velocity from the hydraulic computations for each discharge. The velocity to be used in Equation 5 is the velocity at ½ of the offset height. Based on the predicted velocities at Station

33+00, the stagnation pressure, in feet of water, can be computed for the discharges of 30,000 cfs and 54,000 cfs with specified offset heights. These values are given in Table 2.

**Table 2: Stagnation Pressures at Station 33+00**

| | | | |
|---|---|---|---|
| | k = | **0.001** | **ft** |
| | **Station** | **33+00** | **ft** |
| | | | |
| | **Q** | 30000 | 54000 |
| | ***f*** | 0.0340 | 0.0307 |
| | **Yo** | 1.964 | 3.088 |
| | **Velocity** | 85.508 | 98.578 |
| **Offset, in** | **Offset, ft** | **$P_{stagnation}$, ft** | |
| | | | |
| 0.125 | 0.010 | 27.66 | 28.40 |
| 0.25 | 0.021 | 37.15 | 38.80 |
| 0.5 | 0.042 | 46.63 | 49.19 |
| 1 | 0.083 | 56.12 | 59.59 |
| | | | |
| | **Velocity Distribution** | | |
| | | | |
| | $\frac{v-V}{V\sqrt{f}} = 2\log\left(\frac{y}{Y_o}\right)+0.88$ | | |
| | | | |
| | v = | Velocity | |
| | y = | Depth | |
| | Yo = | Flow Depth | |

Table 2 shows that when the discharge inceased from 30,000- to 54,000 cfs, the stagnation pressure increased by a little more than 6 -percent for the 1-inch offset. Larger offsets would increase even more.

The distribution of the uplift pressures under the slab depend upon the porosity of the foundation material. For example, if the porosity is very low, the uplift pressure would be restricted to the area of the gravel drain. This would be typical for a slab that is well bonded to a rock foundation. On the other hand, if the porosity is very high, the uplift pressure would be almost equal over the entire bottom of the slab. This would be typical for a slab that is place on a very loose gravel foundation. Between these two extremes, the uplift pressure distribution could actually increase above the stagnation pressure if the slab were placed on a steep slope due to the increase in the hydrostatic pressure due to the slope. A detailed study of the seepage flow under the slab would be necessary in this case to determine the uplift pressure distribution.

## 5.0 LEAKAGE RATES

The leakage rate through the joints and cracks in the slab can be estimated for flow through a joint based on the Bernoulli equation, assuming the pressure at the top of the joint or crack is equal to the piezometric pressure and the pressure at the bottom of the joint is equal to atmospheric pressure. The entrance and exit loss coefficients are equal to 0.5 and 1.0 respectively. The friction loss coefficient in the joint is equal to f L/D. Thus, the leakage rate per unit length of joint, q, can be expressed as:

$$q = d\sqrt{\frac{2g(P+T-P_S)}{\left(1+K_i+K_e+\frac{fT}{d}\right)}} \tag{8}$$

In which

d = Gap Opening
f = Darcy-Weisbach Friction Factor
g = Acceleration of Gravity
Ke = Exit Loss Coefficient = 1.0
Ki = Inlet Loss Coefficient = 0.5
P = Piezometric Pressure in Chute
$P_s$ = Pressure in the herringbone drain = 0 when the drains flow freely.
T = Slab Thickness

Equation 8 can be used to estimate the leakage flow through cracks and joints for a free water surface condition under the slab for each of the discharge conditions. The equation assumes the flow is turbulent - that is the Reynolds number is greater than 2,000. The difference in the resistance laws for laminar and turbulent flow is significant The flows with 1/8$^{th}$ inch gaps are in the so called “critical zone” where the flow is neither laminar nor turbulent.

These computations cannot be compared with the tests by Frizell [B-3], because the approach flow velocity distributions and flow depths are completely different in the model test by Frizell [B-3] and in the prototype, shown in Figure B-11. In addition, the sealed chamber conditions tested by Frizell represent a fully saturated foundation so the discharge values through the cracks and joints are not comparable.

The unit discharges for various gaps with the 30,000 cfs and 54,000 cfs discharges and a 7-inch-thick slab are shown in Table 3. These flows represent the flows into the herringbone drains when they are discharging freely. The bottoms of the drains have slots, so part of the water will flow into the foundation material if there is not a tight bond between the slab and the underlying rock and part of the flow will be discharged to the collector drain.

**Table 3: Leakage Rate at Station 33+00**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | **7-Inch thick slab** | | | | | | |
| | | | | | | | | | | |
| | **Q** | **30000** | **54000** | | | | | | | |
| | ***f*** | 0.0340 | 0.0307 | | **Determination of Friction Factor in gap** | | | | | |
| | **Yo** | 1.964 | 3.088 | | | | | | | |
| | **Velocity** | 85.508 | 98.578 | | **Q = 30,000 cfs** | | | **Q = 54,000 cfs** | | |
| **Gap, in** | **Gap, ft** | **Seepage, cfs/ft** | | **Dq/Dv** | | | | | | |
| | | **q** | **q** | | ***f*** | **Reynolds** | **Colebrook** | ***f*** | **Reynolds** | **Colebrook** |
| | | | | | | | | | | |
| 0.125 | 0.010 | 0.03 | 0.04 | 0.001 | 0.0797 | 2.24E+03 | 0.00 | 0.0782 | 2.72E+03 | 0.00 |
| 0.25 | 0.021 | 0.09 | 0.11 | 0.001 | 0.0570 | 6.70E+03 | 0.00 | 0.0562 | 8.08E+03 | 0.00 |
| 0.5 | 0.042 | 0.25 | 0.30 | 0.004 | 0.0432 | 1.77E+04 | 0.00 | 0.0427 | 2.13E+04 | 0.00 |
| 1 | 0.083 | 0.59 | 0.70 | 0.009 | 0.0340 | 4.16E+04 | 0.00 | 0.0337 | 5.00E+04 | 0.00 |

**Poiseuille Flow**

$$q = \frac{gD^3Y_o}{12\nu T}$$

**Turbulent flow for Reynolds > 2E+03**

**Reynolds**

$$R = \frac{VD}{\nu} = \frac{qD}{D\nu} = \frac{q}{\nu}$$

**Colebrook Equation**

$$1.74 - 2\,log\left(\frac{2k}{D} + \frac{18.7}{R\sqrt{f}}\right) - \frac{1}{\sqrt{f}} = 0$$

V = Mean Velocity
q = Unit Discharge
v = Kinematic Viscosity

k = Sand Grain Roughness
D = Gap Thickness
R = Reynolds Number
$f$ = Friction Factor

The collector drains exit into an outfall line as shown in Figure B-15. This line has a maximum capacity before it flows full that is determined by its diameter, friction factor, and invert slope. If the capacity of this outfall line is exceeded, the collector drains will begin to fill and discharge water into the backfill material behind the wall through perforations on the top of the collector pipes. For even higher inflow discharges, the flow in the herringbone drains will begin to fill and pressurize the area under the floor slabs on the chute.

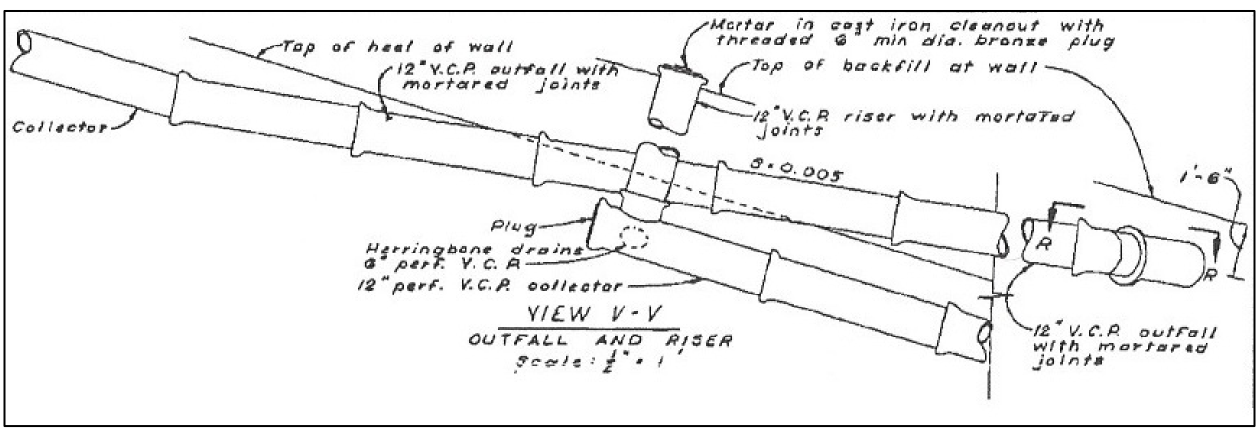


**Figure B-15: Collector Drain and Outfall, Reference [B-8]**

The maximum discharge in the drain outfall line is given in Table 3. Note that at a uniform flow depth of y/D = 0.83, the conduit can either flow full or part full. Small waves on the surface will initiate the transition to full flow. For higher discharges than that given in Table 4, the conduit will flow full.

**Table 4: Maximum Discharge into Drain Outfall**

| | | | | |
|---|---|---|---|---|
| y/D = | 0.83 | $Qn/D^{8/3}S^{1/2}$ = | 0.463 | |
| | | | | |
| | S = | 0.005 | | |
| | D = | 0.9975 | ft | |
| | n = | 0.01 | | |
| | Q = | 3.25 | cfs | |
| | $Q/D^{2.5}$ = | 3.27 | | |
| | Yc/D = | 0.775 | | |
| | Yc = | 0.77 | ft | |

For example, with a 1/4-inch gap in a 7-inch thick slab, the total leakage into the 66.8-foot long herringbone drain would be equal to 0.03 cfs/ft x 66.8 ft = 2 cfs. Since 10 herringbone drains flow into each collector pipe, the total flow into the collector pipe would be 20 cfs. The maximum capacity of the collector pipe before it becomes pressurized is 3.25 cfs. Thus, flows into even very small gaps can exceed the capacity of the collector pipes. This implies that unless the cracks are sealed, all the herringbone drains are probably full of water and that the drain envelopes are saturated.

## 5.1 Observations

The implications of these findings are significant. If the collector drain is flowing at less than its maximum capacity, then the pressure in the herringbone drain will be zero and the leakage rate through the cracks can be predicted reasonably accurately. Part of the leakage flow will be discharged to the collector and part will enter the foundation material if the bond between the foundation rock and the slab is not tight. The water enters the foundation material because the herringbone drains have slots on the invert of the drains. However, if the water flow into the collector drain exceeds its capacity, then the flow under the slab can become saturated and subjected to a pressure that is only slightly less than the stagnation pressure. This sudden increase in pressure as the foundation material becomes saturated could be enough to fail the anchorage causing the upstream end of the slab to move into the flow. As the slab moves into the flow, the stagnation pressures will increase even further as shown in Table 1. However, as this process is happening, the leakage into the foundation will significantly decrease because the pressure term under the slab, Ps, in equation 8 will become positive.

## 6.0 SUMMARY

1. Aeration of the flow was due to turbulence at the water surface and for most flows from roll waves on the surface. Aeration due to the formation of roll waves on spillway flows is not common and is an excellent topic for further research by the hydraulic community.
2. Cavitation was not a contributor to the failure of the service spillway chute. Both numerical computations and on-site examination of the remaining downstream chute validate this observation.
3. The pressures under the slab increases in proportion to the amount that the slab is offset into the flow. Thus, if a slab starts to move into the flow, the pressures under the slab will increase exacerbating the deflection of the slab into the flow. By analogy, the pressures on the downstream face of any repair pop-outs increases in proportion to the depth of the pop-out.
4. The pressures under the slab also increase in proportion to the rate of flow in the chute or the velocity squared.
5. The leakage rates through the joints and cracks over the herringbone drains are significant. These rates were large enough to saturate the foundation material under the slabs and to exceed the capacity of the collector drains.

## 7.0 REFERENCES

[B-1] Falvey, H. T., 1990, USBR Engineering Monograph 42, Cavitation in Chutes and Spillways.

[B-2] Johnson, P.L., 1976, "Research into Uplift on Steep Chute Lateral Linings," USBR PAP 1163.

[B-3] Frizell, K.W., 2007, "Uplift and Crack Flow Resulting from High Velocity Discharges Over Open Offset Joints" USBR Report DSO-07-07.

[B-4] Mercury News. http://www.mercurynews.com/2017/03/11/oroville-dam-photos-taken-weeks-before-spillway-broke-show-something-wrong/.

[B-5] California Department of Water Resources (2017a): Orville Spillway Incident, (http://www.water.ca.gov/oroville-spillway/index.cfm).

[B-6] Design of Small Canal Structures, US Bureau of Reclamation, Figure 2-45.

[B-7] Oroville Dam DSOD Report 2014 – 2015, OFD-ORO-2015_11_18-386.

[B-8] Oroville Dam Chute Drawing A-3B9-1, Sheet No. 47, Feb 26, 1965.

# ATTACHMENT

# HYDRAULIC AND CAVITATION COMPUTATIONS

# FOR SELECTED DISCHARGES

# DISCHARGE = 30,000 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| ft³/s | ft | ft | | ft |
| 30000 | 3.709731 | 0.0010000 | 0.012378 | 850.0005 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 3.7097 | 45.262 | 3.691 | 847.384 | 0.00 | S2 | 2.426 | 9.596 | 4.16 | 0.755 | |
| 1350.000 | 810.34 | 0.0282 | 3.7082 | 45.281 | 3.617 | 846.076 | 0.00 | S2 | 3.621 | 9.585 | 4.16 | 1.103 | |
| 1400.000 | 807.99 | 0.0470 | 3.6598 | 45.879 | 3.565 | 844.739 | 0.00 | S2 | 3.079 | 9.607 | 4.26 | 1.431 | |
| 1500.000 | 802.32 | 0.0567 | 3.5441 | 47.378 | 3.538 | 841.863 | 0.00 | S2 | 2.902 | 9.675 | 4.51 | 2.032 | |
| 1600.000 | 796.66 | 0.0567 | 3.4608 | 48.518 | 3.455 | 838.725 | 0.00 | S2 | 2.902 | 9.748 | 4.73 | 2.597 | |
| 1700.000 | 790.99 | 0.0567 | 3.4039 | 49.329 | 3.398 | 835.383 | 0.00 | S2 | 2.902 | 9.835 | 4.91 | 3.136 | |
| 1800.000 | 785.33 | 0.0567 | 3.3393 | 50.283 | 3.334 | 831.847 | 0.00 | S2 | 2.902 | 9.880 | 5.09 | 3.339 | |
| 1900.000 | 779.66 | 0.0567 | 3.2658 | 51.415 | 3.261 | 828.073 | 0.00 | S2 | 2.902 | 9.879 | 5.26 | 3.266 | |
| 2000.000 | 773.99 | 0.0567 | 3.2059 | 52.375 | 3.201 | 824.050 | 0.00 | S2 | 2.902 | 9.879 | 5.41 | 3.206 | |
| 2100.000 | 768.33 | 0.0567 | 3.1567 | 53.192 | 3.152 | 819.807 | 0.00 | S2 | 2.902 | 9.878 | 5.54 | 3.157 | |
| 2200.000 | 762.66 | 0.0567 | 3.1160 | 53.886 | 3.111 | 815.371 | 0.00 | S2 | 2.902 | 9.881 | 5.65 | 3.116 | |
| 2300.000 | 757.00 | 0.0566 | 3.0823 | 54.475 | 3.077 | 810.765 | 0.00 | S2 | 2.902 | 9.881 | 5.74 | 3.082 | |
| 2400.000 | 749.76 | 0.0724 | 3.0079 | 55.823 | 2.901 | 805.887 | 0.00 | S2 | 2.686 | 9.874 | 5.95 | 3.008 | |
| 2500.000 | 739.39 | 0.1037 | 2.8747 | 58.409 | 2.756 | 800.419 | 0.00 | S2 | 2.399 | 9.883 | 6.38 | 2.875 | |
| 2600.000 | 725.88 | 0.1351 | 2.7118 | 61.919 | 2.578 | 793.945 | 0.00 | S2 | 2.209 | 9.895 | 6.98 | 2.712 | |
| 2700.000 | 709.23 | 0.1665 | 2.5437 | 66.010 | 2.394 | 786.051 | 0.00 | S2 | 2.071 | 9.911 | 7.70 | 2.544 | Probable |
| 2800.000 | 689.44 | 0.1979 | 2.3849 | 70.405 | 2.219 | 776.326 | 0.00 | S2 | 1.965 | 9.929 | 8.50 | 2.385 | Probable |
| 2900.000 | 666.52 | 0.2292 | 2.2420 | 74.892 | 2.060 | 764.382 | 0.00 | S2 | 1.880 | 9.949 | 9.36 | 2.242 | Probable |
| 3000.000 | 642.03 | 0.2450 | 2.1315 | 78.774 | 2.070 | 750.088 | 0.00 | S2 | 1.843 | 9.969 | 10.12 | 2.132 | Probable |
| 3100.000 | 617.53 | 0.2450 | 2.0538 | 81.754 | 1.995 | 733.689 | 0.00 | S2 | 1.843 | 9.972 | 10.70 | 2.054 | Probable |
| 3200.000 | 593.04 | 0.2450 | 1.9986 | 84.015 | 1.941 | 715.542 | 0.00 | S2 | 1.843 | 9.972 | 11.15 | 1.999 | Probable |
| 3300.000 | 568.54 | 0.2450 | 1.9586 | 85.732 | 1.902 | 695.984 | 0.00 | S2 | 1.843 | 9.972 | 11.49 | 1.959 | Probable |
| 3400.000 | 544.05 | 0.2450 | 1.9292 | 87.035 | 1.874 | 675.306 | 0.00 | S2 | 1.843 | 9.972 | 11.75 | 1.929 | Probable |
| 3500.000 | 519.55 | 0.2450 | 1.9076 | 88.023 | 1.853 | 653.746 | 0.00 | S2 | 1.843 | 9.972 | 11.95 | 1.908 | Probable |
| 3600.000 | 495.06 | 0.2450 | 1.8914 | 88.774 | 1.837 | 631.501 | 0.00 | S2 | 1.843 | 9.972 | 12.11 | 1.891 | Probable |
| 3700.000 | 470.56 | 0.2450 | 1.8794 | 89.343 | 1.825 | 608.726 | 0.00 | S2 | 1.843 | 9.972 | 12.22 | 1.879 | Probable |
| 3800.000 | 446.07 | 0.2450 | 1.8704 | 89.774 | 1.817 | 585.543 | 0.00 | S2 | 1.843 | 9.972 | 12.31 | 1.870 | Probable |
| 3900.000 | 421.57 | 0.2450 | 1.8636 | 90.102 | 1.810 | 562.047 | 0.00 | S2 | 1.843 | 9.972 | 12.38 | 1.864 | Probable |
| 4000.000 | 397.08 | 0.2450 | 1.8584 | 90.350 | 1.805 | 538.311 | 0.00 | S2 | 1.843 | 9.972 | 12.43 | 1.858 | Probable |
| 4100.000 | 372.58 | 0.2450 | 1.8546 | 90.538 | 1.801 | 514.393 | 0.00 | S2 | 1.843 | 9.972 | 12.47 | 1.855 | Probable |
| 4200.000 | 348.09 | 0.2450 | 1.8517 | 90.680 | 1.799 | 490.337 | 0.00 | S2 | 1.843 | 9.972 | 12.50 | 1.852 | Probable |
| 4300.000 | 323.59 | 0.2450 | 1.8495 | 90.788 | 1.796 | 466.174 | 0.00 | S2 | 1.843 | 9.972 | 12.52 | 1.849 | Probable |
| 4355.000 | 310.12 | 0.2450 | 1.8485 | 90.836 | 1.795 | 452.849 | 0.00 | S2 | 1.843 | 9.972 | 12.53 | 1.848 | Probable |

## COMPUTED FLOW PROFILE - CAVITATION PROPERTIES

Compute Flow Profile & Cavitation Output

| OROVILLE DAM | | | |
|---|---|---|---|
| Q | Initial Depth | Rugosity (ft or m) | n |
| ft³/s | ft | ft | |
| 30000 | 3.709730577 | 0.001 | 0.012378 |

| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | DAMAGE POTENTIAL | | | | | | Turbulence Intensity | Stream Power |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Circular Arc 1/4-in (5-mm) | Circular Arc 1/2-in (12.5-mm) | Circular Arc 1-in (25-mm) | 90° Offset 1/4-in (5-mm) | 90° Offset 1/2-in (12.5-mm) | 90° Offset 1-in (25-mm) | | |
| ft | | | n : 1 | | | | | | | | kW/m² |
| 1300.00 | 1.138 | 0.047 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0.032 | 4.03 |
| 1350.00 | 1.135 | 0.047 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0.030 | 4.03 |
| 1400.00 | 1.104 | 0.047 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.030 | 4.20 |
| 1500.00 | 1.035 | 0.047 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.029 | 4.65 |
| 1600.00 | 0.984 | 0.047 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.028 | 5.01 |
| 1700.00 | 0.951 | 0.048 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.027 | 5.28 |
| 1800.00 | 0.913 | 0.048 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.027 | 5.61 |
| 1900.00 | 0.872 | 0.048 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.027 | 6.01 |
| 2000.00 | 0.839 | 0.048 | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0.027 | 6.37 |
| 2100.00 | 0.812 | 0.048 | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0.027 | 6.69 |
| 2200.00 | 0.790 | 0.048 | 4 | 0 | 0 | 0 | 0 | 0 | 1 | 0.027 | 6.97 |
| 2300.00 | 0.773 | 0.048 | 4 | 0 | 0 | 0 | 0 | 0 | 1 | 0.027 | 7.21 |
| 2400.00 | 0.732 | 0.049 | 4 | 0 | 0 | 1 | 0 | 0 | 1 | 0.027 | 7.79 |
| 2500.00 | 0.666 | 0.049 | 5 | 0 | 0 | 1 | 0 | 0 | 2 | 0.028 | 8.98 |
| 2600.00 | 0.590 | 0.050 | 5 | 0 | 1 | 3 | 0 | 1 | 4 | 0.028 | 10.80 |
| 2700.00 | 0.516 | 0.050 | 6 | 0 | 2 | 6 | 1 | 3 | 9 | 0.028 | 13.22 |
| 2800.00 | 0.451 | 0.051 | 8 | 2 | 5 | 13 | 2 | 7 | 19 | 0.028 | 16.20 |
| 2900.00 | 0.397 | 0.051 | 9 | 4 | 11 | 27 | 5 | 15 | 38 | 0.028 | 19.70 |
| 3000.00 | 0.359 | 0.052 | 11 | 7 | 19 | 45 | 10 | 26 | 63 | 0.028 | 23.12 |
| 3100.00 | 0.333 | 0.052 | 12 | 11 | 28 | 66 | 15 | 39 | 93 | 0.028 | 26.00 |
| 3200.00 | 0.314 | 0.052 | 13 | 16 | 38 | 88 | 21 | 52 | 123 | 0.029 | 28.35 |
| 3300.00 | 0.302 | 0.053 | 13 | 20 | 47 | 108 | 26 | 64 | 151 | 0.029 | 30.23 |
| 3400.00 | 0.292 | 0.053 | 14 | 23 | 55 | 126 | 31 | 75 | 175 | 0.029 | 31.71 |
| 3500.00 | 0.286 | 0.053 | 14 | 26 | 62 | 141 | 35 | 84 | 196 | 0.029 | 32.87 |
| 3600.00 | 0.281 | 0.053 | 15 | 29 | 68 | 153 | 38 | 92 | 214 | 0.029 | 33.76 |
| 3700.00 | 0.277 | 0.053 | 15 | 31 | 73 | 164 | 41 | 99 | 228 | 0.029 | 34.46 |
| 3800.00 | 0.274 | 0.053 | 15 | 33 | 76 | 172 | 43 | 104 | 239 | 0.029 | 34.99 |
| 3900.00 | 0.272 | 0.053 | 15 | 34 | 79 | 178 | 45 | 107 | 248 | 0.029 | 35.39 |
| 4000.00 | 0.271 | 0.053 | 16 | 35 | 82 | 183 | 46 | 111 | 255 | 0.029 | 35.70 |
| 4100.00 | 0.270 | 0.053 | 16 | 36 | 83 | 187 | 47 | 113 | 260 | 0.029 | 35.94 |
| 4200.00 | 0.269 | 0.053 | 16 | 36 | 85 | 190 | 48 | 115 | 265 | 0.029 | 36.12 |
| 4300.00 | 0.268 | 0.053 | 16 | 37 | 86 | 192 | 49 | 116 | 268 | 0.029 | 36.26 |
| 4355.00 | 0.268 | 0.053 | 16 | 37 | 86 | 193 | 49 | 117 | 269 | 0.029 | 36.32 |

# DISCHARGE = 54,500 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| $ft^3/s$ | ft | ft | | ft |
| 54500 | 5.779408 | 0.0010000 | 0.01277 | 863.0002 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 5.7794 | 52.780 | 5.751 | 860.827 | 0.00 | S2 | 3.543 | 14.275 | 3.88 | 0.732 | |
| 1350.000 | 810.34 | 0.0282 | 5.7507 | 53.043 | 5.559 | 859.740 | 0.00 | S2 | 5.302 | 14.236 | 3.90 | 1.069 | |
| 1400.000 | 807.99 | 0.0470 | 5.6735 | 53.765 | 5.475 | 858.620 | 0.00 | S2 | 4.502 | 14.251 | 3.99 | 1.387 | |
| 1500.000 | 802.32 | 0.0567 | 5.4985 | 55.476 | 5.490 | 856.216 | 0.00 | S2 | 4.242 | 14.312 | 4.20 | 1.970 | |
| 1600.000 | 796.66 | 0.0567 | 5.3507 | 57.009 | 5.342 | 853.582 | 0.00 | S2 | 4.242 | 14.355 | 4.39 | 2.517 | |
| 1700.000 | 790.99 | 0.0567 | 5.2338 | 58.282 | 5.225 | 850.740 | 0.00 | S2 | 4.242 | 14.409 | 4.57 | 3.038 | |
| 1800.000 | 785.33 | 0.0567 | 5.1410 | 59.334 | 5.133 | 847.715 | 0.00 | S2 | 4.242 | 14.472 | 4.72 | 3.538 | |
| 1900.000 | 779.66 | 0.0567 | 5.0672 | 60.198 | 5.059 | 844.534 | 0.00 | S2 | 4.242 | 14.543 | 4.86 | 4.022 | |
| 2000.000 | 773.99 | 0.0567 | 5.0086 | 60.903 | 5.001 | 841.222 | 0.00 | S2 | 4.242 | 14.622 | 4.99 | 4.492 | |
| 2100.000 | 768.33 | 0.0567 | 4.9621 | 61.473 | 4.954 | 837.801 | 0.00 | S2 | 4.242 | 14.707 | 5.10 | 4.950 | |
| 2200.000 | 762.66 | 0.0567 | 4.8813 | 62.491 | 4.873 | 834.239 | 0.00 | S2 | 4.242 | 14.709 | 5.23 | 4.881 | |
| 2300.000 | 757.00 | 0.0566 | 4.8099 | 63.418 | 4.802 | 830.499 | 0.00 | S2 | 4.243 | 14.708 | 5.35 | 4.810 | |
| 2400.000 | 749.76 | 0.0724 | 4.6890 | 65.053 | 4.467 | 826.511 | 0.00 | S2 | 3.924 | 14.690 | 5.56 | 4.689 | |
| 2500.000 | 739.39 | 0.1037 | 4.5018 | 67.759 | 4.259 | 822.072 | 0.00 | S2 | 3.502 | 14.707 | 5.91 | 4.502 | |
| 2600.000 | 725.88 | 0.1351 | 4.2734 | 71.380 | 4.006 | 816.914 | 0.00 | S2 | 3.223 | 14.725 | 6.41 | 4.273 | |
| 2700.000 | 709.23 | 0.1665 | 4.0316 | 75.662 | 3.738 | 810.750 | 0.00 | S2 | 3.021 | 14.748 | 7.01 | 4.032 | |
| 2800.000 | 689.44 | 0.1979 | 3.7947 | 80.384 | 3.472 | 803.282 | 0.00 | S2 | 2.865 | 14.776 | 7.70 | 3.795 | |
| 2900.000 | 666.52 | 0.2292 | 3.5735 | 85.361 | 3.223 | 794.202 | 0.00 | S2 | 2.740 | 14.807 | 8.45 | 3.573 | Probable |
| 3000.000 | 642.03 | 0.2450 | 3.3933 | 89.895 | 3.296 | 783.353 | 0.00 | S2 | 2.686 | 14.845 | 9.15 | 3.393 | Probable |
| 3100.000 | 617.53 | 0.2450 | 3.2539 | 93.745 | 3.160 | 770.797 | 0.00 | S2 | 2.686 | 14.844 | 9.75 | 3.254 | Probable |
| 3200.000 | 593.04 | 0.2450 | 3.1461 | 96.958 | 3.056 | 756.666 | 0.00 | S2 | 2.686 | 14.842 | 10.25 | 3.146 | Probable |
| 3300.000 | 568.54 | 0.2450 | 3.0612 | 99.647 | 2.973 | 741.117 | 0.00 | S2 | 2.686 | 14.847 | 10.68 | 3.061 | Probable |
| 3400.000 | 544.05 | 0.2450 | 2.9935 | 101.899 | 2.908 | 724.308 | 0.00 | S2 | 2.686 | 14.847 | 11.05 | 2.994 | Probable |
| 3500.000 | 519.55 | 0.2450 | 2.9391 | 103.786 | 2.855 | 706.389 | 0.00 | S2 | 2.686 | 14.847 | 11.35 | 2.939 | Probable |
| 3600.000 | 495.06 | 0.2450 | 2.8950 | 105.368 | 2.812 | 687.503 | 0.00 | S2 | 2.686 | 14.847 | 11.61 | 2.895 | Probable |
| 3700.000 | 470.56 | 0.2450 | 2.8590 | 106.694 | 2.777 | 667.777 | 0.00 | S2 | 2.686 | 14.847 | 11.83 | 2.859 | Probable |
| 3800.000 | 446.07 | 0.2450 | 2.8295 | 107.806 | 2.748 | 647.328 | 0.00 | S2 | 2.686 | 14.847 | 12.02 | 2.829 | Probable |
| 3900.000 | 421.57 | 0.2450 | 2.8052 | 108.739 | 2.725 | 626.259 | 0.00 | S2 | 2.686 | 14.847 | 12.18 | 2.805 | Probable |
| 4000.000 | 397.08 | 0.2450 | 2.7852 | 109.521 | 2.705 | 604.660 | 0.00 | S2 | 2.686 | 14.847 | 12.31 | 2.785 | Probable |
| 4100.000 | 372.58 | 0.2450 | 2.7686 | 110.176 | 2.689 | 582.608 | 0.00 | S2 | 2.686 | 14.847 | 12.42 | 2.769 | Probable |
| 4200.000 | 348.09 | 0.2450 | 2.7549 | 110.726 | 2.676 | 560.174 | 0.00 | S2 | 2.686 | 14.847 | 12.51 | 2.755 | Probable |
| 4300.000 | 323.59 | 0.2450 | 2.7435 | 111.186 | 2.665 | 537.414 | 0.00 | S2 | 2.686 | 14.847 | 12.59 | 2.743 | Probable |
| 4355.000 | 310.12 | 0.2450 | 2.7380 | 111.407 | 2.659 | 524.775 | 0.00 | S2 | 2.686 | 14.847 | 12.63 | 2.738 | Probable |

## COMPUTED FLOW PROFILE - CAVITATION PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | Initial Depth | Rugosity (ft or m) | n |
|---|---|---|---|
| $ft^3/s$ | ft | ft | |
| 54500 | 5.78 | 0.001 | 0.01277 |

| | | | | DAMAGE POTENTIAL | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Circular Arc | | | 90° Offset | | | | |
| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | 1/4-in (5-mm) | 1/2-in (12.5-mm) | 1-in (25-mm) | 1/4-in (5-mm) | 1/2-in (12.5-mm) | 1-in (25-mm) | Turbulence Intensity | Stream Power |
| ft | | | n : 1 | | | | | | | | $kW/m^2$ |
| 1300.00 | 0.885 | 0.043 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.032 | 6.04 |
| 1350.00 | 0.872 | 0.043 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.030 | 6.14 |
| 1400.00 | 0.847 | 0.043 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.030 | 6.40 |
| 1500.00 | 0.795 | 0.044 | 4 | 0 | 0 | 0 | 0 | 0 | 1 | 0.029 | 7.05 |
| 1600.00 | 0.750 | 0.044 | 4 | 0 | 0 | 1 | 0 | 0 | 1 | 0.028 | 7.68 |
| 1700.00 | 0.716 | 0.044 | 4 | 0 | 0 | 1 | 0 | 0 | 1 | 0.027 | 8.22 |
| 1800.00 | 0.689 | 0.044 | 4 | 0 | 0 | 1 | 0 | 0 | 1 | 0.027 | 8.69 |
| 1900.00 | 0.668 | 0.044 | 5 | 0 | 0 | 1 | 0 | 0 | 2 | 0.027 | 9.09 |
| 2000.00 | 0.652 | 0.044 | 5 | 0 | 0 | 1 | 0 | 0 | 2 | 0.026 | 9.43 |
| 2100.00 | 0.639 | 0.044 | 5 | 0 | 0 | 1 | 0 | 0 | 2 | 0.026 | 9.71 |
| 2200.00 | 0.617 | 0.045 | 5 | 0 | 0 | 2 | 0 | 1 | 2 | 0.026 | 10.22 |
| 2300.00 | 0.598 | 0.045 | 5 | 0 | 1 | 2 | 0 | 1 | 3 | 0.026 | 10.70 |
| 2400.00 | 0.563 | 0.045 | 6 | 0 | 1 | 3 | 0 | 1 | 4 | 0.026 | 11.58 |
| 2500.00 | 0.516 | 0.045 | 6 | 0 | 2 | 5 | 0 | 2 | 6 | 0.026 | 13.16 |
| 2600.00 | 0.462 | 0.046 | 7 | 1 | 4 | 10 | 1 | 4 | 12 | 0.027 | 15.48 |
| 2700.00 | 0.408 | 0.046 | 9 | 3 | 8 | 20 | 3 | 9 | 24 | 0.027 | 18.58 |
| 2800.00 | 0.359 | 0.047 | 11 | 6 | 16 | 39 | 7 | 19 | 46 | 0.027 | 22.46 |
| 2900.00 | 0.316 | 0.047 | 13 | 13 | 32 | 75 | 14 | 36 | 88 | 0.027 | 27.12 |
| 3000.00 | 0.285 | 0.047 | 14 | 23 | 55 | 124 | 25 | 62 | 146 | 0.027 | 31.91 |
| 3100.00 | 0.262 | 0.048 | 16 | 36 | 85 | 191 | 40 | 96 | 224 | 0.027 | 36.41 |
| 3200.00 | 0.244 | 0.048 | 18 | 53 | 121 | 269 | 58 | 137 | 315 | 0.027 | 40.48 |
| 3300.00 | 0.230 | 0.048 | 19 | 71 | 160 | 354 | 77 | 182 | 415 | 0.027 | 44.12 |
| 3400.00 | 0.220 | 0.049 | 21 | 89 | 201 | 442 | 98 | 229 | 519 | 0.027 | 47.34 |
| 3500.00 | 0.212 | 0.049 | 22 | 108 | 243 | 531 | 119 | 276 | 624 | 0.028 | 50.16 |
| 3600.00 | 0.205 | 0.049 | 23 | 127 | 283 | 616 | 139 | 321 | 725 | 0.028 | 52.61 |
| 3700.00 | 0.200 | 0.049 | 24 | 144 | 321 | 698 | 158 | 365 | 821 | 0.028 | 54.73 |
| 3800.00 | 0.195 | 0.049 | 24 | 161 | 356 | 773 | 176 | 405 | 909 | 0.028 | 56.55 |
| 3900.00 | 0.192 | 0.049 | 25 | 176 | 388 | 841 | 193 | 441 | 990 | 0.028 | 58.10 |
| 4000.00 | 0.189 | 0.049 | 26 | 189 | 417 | 903 | 207 | 474 | 1063 | 0.028 | 59.43 |
| 4100.00 | 0.187 | 0.049 | 26 | 201 | 443 | 957 | 220 | 503 | 1127 | 0.028 | 60.56 |
| 4200.00 | 0.185 | 0.049 | 26 | 211 | 465 | 1005 | 232 | 529 | 1184 | 0.028 | 61.52 |
| 4300.00 | 0.183 | 0.049 | 27 | 221 | 485 | 1047 | 242 | 552 | 1233 | 0.028 | 62.33 |
| 4355.00 | 0.183 | 0.049 | 27 | 225 | 495 | 1068 | 247 | 563 | 1257 | 0.028 | 62.72 |

# DISCHARGE = 100,000 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| ft³/s | ft | ft | | ft |
| 100000 | 7.86887 | 0.0010000 | 0.013052 | 901.0017 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 7.8689 | 71.128 | 7.830 | 898.200 | 0.00 | S2 | 5.220 | 21.388 | 4.48 | 0.690 | |
| 1350.000 | 810.34 | 0.0282 | 7.8495 | 71.304 | 7.381 | 896.799 | 0.00 | S2 | 7.844 | 21.305 | 4.48 | 1.008 | |
| 1400.000 | 807.99 | 0.0470 | 7.8056 | 71.705 | 7.328 | 895.380 | 0.00 | S2 | 6.649 | 21.318 | 4.53 | 1.308 | |
| 1500.000 | 802.32 | 0.0567 | 7.7038 | 72.653 | 7.691 | 892.456 | 0.00 | S2 | 6.260 | 21.400 | 4.63 | 1.860 | |
| 1600.000 | 796.66 | 0.0567 | 7.5946 | 73.697 | 7.582 | 889.405 | 0.00 | S2 | 6.260 | 21.428 | 4.74 | 2.379 | |
| 1700.000 | 790.99 | 0.0567 | 7.5001 | 74.626 | 7.488 | 886.227 | 0.00 | S2 | 6.260 | 21.462 | 4.84 | 2.875 | |
| 1800.000 | 785.33 | 0.0567 | 7.4183 | 75.449 | 7.406 | 882.930 | 0.00 | S2 | 6.260 | 21.502 | 4.94 | 3.352 | |
| 1900.000 | 779.66 | 0.0567 | 7.3475 | 76.176 | 7.336 | 879.527 | 0.00 | S2 | 6.260 | 21.547 | 5.02 | 3.813 | |
| 2000.000 | 773.99 | 0.0567 | 7.2863 | 76.816 | 7.275 | 876.029 | 0.00 | S2 | 6.260 | 21.597 | 5.10 | 4.262 | |
| 2100.000 | 768.33 | 0.0567 | 7.2333 | 77.378 | 7.222 | 872.445 | 0.00 | S2 | 6.260 | 21.652 | 5.18 | 4.700 | |
| 2200.000 | 762.66 | 0.0567 | 7.1877 | 77.869 | 7.176 | 868.785 | 0.00 | S2 | 6.260 | 21.712 | 5.25 | 5.127 | |
| 2300.000 | 757.00 | 0.0566 | 7.1486 | 78.296 | 7.137 | 865.058 | 0.00 | S2 | 6.262 | 21.775 | 5.32 | 5.547 | |
| 2400.000 | 749.76 | 0.0724 | 7.0494 | 79.397 | 6.574 | 861.212 | 0.00 | S2 | 5.787 | 21.809 | 5.46 | 5.958 | |
| 2500.000 | 739.39 | 0.1037 | 6.8855 | 81.286 | 6.374 | 857.125 | 0.00 | S2 | 5.160 | 21.928 | 5.70 | 6.362 | |
| 2600.000 | 725.88 | 0.1351 | 6.6594 | 84.047 | 6.106 | 852.642 | 0.00 | S2 | 4.746 | 22.053 | 6.04 | 6.659 | |
| 2700.000 | 709.23 | 0.1665 | 6.3536 | 88.092 | 5.756 | 847.535 | 0.00 | S2 | 4.446 | 22.088 | 6.50 | 6.354 | |
| 2800.000 | 689.44 | 0.1979 | 6.0389 | 92.682 | 5.394 | 841.557 | 0.00 | S2 | 4.214 | 22.129 | 7.03 | 6.039 | |
| 2900.000 | 666.52 | 0.2292 | 5.7306 | 97.668 | 5.039 | 834.495 | 0.00 | S2 | 4.030 | 22.177 | 7.63 | 5.731 | |
| 3000.000 | 642.03 | 0.2450 | 5.4690 | 102.340 | 5.312 | 826.232 | 0.00 | S2 | 3.950 | 22.251 | 8.21 | 5.469 | |
| 3100.000 | 617.53 | 0.2450 | 5.2499 | 106.612 | 5.099 | 816.770 | 0.00 | S2 | 3.950 | 22.250 | 8.73 | 5.250 | |
| 3200.000 | 593.04 | 0.2450 | 5.0699 | 110.396 | 4.924 | 806.128 | 0.00 | S2 | 3.950 | 22.249 | 9.19 | 5.070 | |
| 3300.000 | 568.54 | 0.2450 | 4.9201 | 113.758 | 4.779 | 794.358 | 0.00 | S2 | 3.950 | 22.248 | 9.62 | 4.920 | |
| 3400.000 | 544.05 | 0.2450 | 4.7940 | 116.749 | 4.656 | 781.519 | 0.00 | S2 | 3.950 | 22.246 | 10.00 | 4.794 | |
| 3500.000 | 519.55 | 0.2450 | 4.6870 | 119.415 | 4.552 | 767.672 | 0.00 | S2 | 3.950 | 22.252 | 10.34 | 4.687 | |
| 3600.000 | 495.06 | 0.2450 | 4.5955 | 121.792 | 4.464 | 752.882 | 0.00 | S2 | 3.950 | 22.252 | 10.65 | 4.596 | |
| 3700.000 | 470.56 | 0.2450 | 4.5169 | 123.914 | 4.387 | 737.215 | 0.00 | S2 | 3.950 | 22.252 | 10.93 | 4.517 | |
| 3800.000 | 446.07 | 0.2450 | 4.4488 | 125.808 | 4.321 | 720.734 | 0.00 | S2 | 3.950 | 22.252 | 11.19 | 4.449 | |
| 3900.000 | 421.57 | 0.2450 | 4.3898 | 127.500 | 4.264 | 703.502 | 0.00 | S2 | 3.950 | 22.252 | 11.41 | 4.390 | |
| 4000.000 | 397.08 | 0.2450 | 4.3384 | 129.012 | 4.214 | 685.580 | 0.00 | S2 | 3.950 | 22.252 | 11.62 | 4.338 | |
| 4100.000 | 372.58 | 0.2450 | 4.2934 | 130.362 | 4.170 | 667.026 | 0.00 | S2 | 3.950 | 22.252 | 11.80 | 4.293 | |
| 4200.000 | 348.09 | 0.2450 | 4.2540 | 131.569 | 4.132 | 647.893 | 0.00 | S2 | 3.950 | 22.252 | 11.96 | 4.254 | |
| 4300.000 | 323.59 | 0.2450 | 4.2194 | 132.648 | 4.098 | 628.234 | 0.00 | S2 | 3.950 | 22.252 | 12.11 | 4.219 | |
| 4355.000 | 310.12 | 0.2450 | 4.2022 | 133.192 | 4.082 | 617.212 | 0.00 | S2 | 3.950 | 22.252 | 12.19 | 4.202 | Probable |

## COMPUTED FLOW PROFILE - CAVITATION PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | Initial Depth | Rugosity (ft or m) | n |
|---|---|---|---|
| ft³/s | ft | ft | |
| 100000 | 7.87 | 0.001 | 0.013052 |

| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | DAMAGE POTENTIAL: Circular Arc 1/4-in (5-mm) | Circular Arc 1/2-in (12.5-mm) | Circular Arc 1-in (25-mm) | 90° Offset 1/4-in (5-mm) | 90° Offset 1/2-in (12.5-mm) | 90° Offset 1-in (25-mm) | Turbulence Intensity | Stream Power |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | | | n : 1 | | | | | | | | kW/m² |
| 1300.00 | 0.514 | 0.041 | 7 | 1 | 4 | 10 | 3 | 9 | 22 | 0.032 | 14.34 |
| 1350.00 | 0.505 | 0.041 | 7 | 1 | 4 | 10 | 2 | 7 | 19 | 0.031 | 14.45 |
| 1400.00 | 0.499 | 0.041 | 7 | 1 | 4 | 10 | 2 | 7 | 18 | 0.030 | 14.71 |
| 1500.00 | 0.491 | 0.041 | 7 | 1 | 4 | 10 | 2 | 6 | 15 | 0.029 | 15.31 |
| 1600.00 | 0.476 | 0.041 | 7 | 1 | 4 | 11 | 2 | 6 | 16 | 0.028 | 16.00 |
| 1700.00 | 0.463 | 0.041 | 7 | 1 | 5 | 12 | 2 | 6 | 16 | 0.028 | 16.63 |
| 1800.00 | 0.452 | 0.041 | 8 | 2 | 5 | 13 | 2 | 6 | 16 | 0.027 | 17.20 |
| 1900.00 | 0.442 | 0.042 | 8 | 2 | 5 | 14 | 2 | 6 | 17 | 0.027 | 17.72 |
| 2000.00 | 0.434 | 0.042 | 8 | 2 | 6 | 15 | 2 | 6 | 17 | 0.027 | 18.18 |
| 2100.00 | 0.428 | 0.042 | 8 | 2 | 6 | 16 | 2 | 7 | 18 | 0.026 | 18.60 |
| 2200.00 | 0.422 | 0.042 | 8 | 2 | 6 | 16 | 2 | 7 | 18 | 0.026 | 18.96 |
| 2300.00 | 0.417 | 0.042 | 9 | 2 | 7 | 17 | 2 | 7 | 18 | 0.026 | 19.29 |
| 2400.00 | 0.399 | 0.042 | 9 | 3 | 8 | 21 | 3 | 8 | 21 | 0.026 | 20.14 |
| 2500.00 | 0.379 | 0.042 | 10 | 4 | 11 | 27 | 3 | 10 | 27 | 0.026 | 21.65 |
| 2600.00 | 0.352 | 0.042 | 11 | 6 | 16 | 38 | 5 | 15 | 37 | 0.026 | 24.01 |
| 2700.00 | 0.318 | 0.043 | 12 | 11 | 27 | 64 | 10 | 26 | 63 | 0.026 | 27.77 |
| 2800.00 | 0.284 | 0.043 | 15 | 20 | 49 | 112 | 18 | 46 | 110 | 0.026 | 32.51 |
| 2900.00 | 0.254 | 0.043 | 17 | 37 | 87 | 195 | 34 | 82 | 193 | 0.026 | 38.25 |
| 3000.00 | 0.233 | 0.044 | 19 | 59 | 135 | 299 | 53 | 128 | 295 | 0.026 | 44.24 |
| 3100.00 | 0.213 | 0.044 | 22 | 92 | 207 | 455 | 84 | 197 | 450 | 0.026 | 50.24 |
| 3200.00 | 0.198 | 0.044 | 24 | 133 | 297 | 648 | 122 | 284 | 643 | 0.026 | 56.01 |
| 3300.00 | 0.186 | 0.044 | 26 | 183 | 405 | 877 | 168 | 387 | 872 | 0.026 | 61.50 |
| 3400.00 | 0.176 | 0.045 | 28 | 240 | 528 | 1138 | 221 | 505 | 1133 | 0.026 | 66.69 |
| 3500.00 | 0.167 | 0.045 | 30 | 303 | 663 | 1425 | 280 | 636 | 1421 | 0.026 | 71.56 |
| 3600.00 | 0.161 | 0.045 | 32 | 372 | 809 | 1734 | 343 | 777 | 1731 | 0.026 | 76.11 |
| 3700.00 | 0.155 | 0.045 | 34 | 444 | 963 | 2058 | 410 | 926 | 2056 | 0.026 | 80.33 |
| 3800.00 | 0.150 | 0.045 | 35 | 518 | 1121 | 2392 | 479 | 1079 | 2391 | 0.026 | 84.23 |
| 3900.00 | 0.146 | 0.045 | 37 | 593 | 1281 | 2729 | 549 | 1234 | 2730 | 0.026 | 87.82 |
| 4000.00 | 0.142 | 0.045 | 38 | 668 | 1441 | 3065 | 620 | 1389 | 3068 | 0.027 | 91.12 |
| 4100.00 | 0.139 | 0.045 | 39 | 743 | 1598 | 3396 | 689 | 1542 | 3401 | 0.027 | 94.14 |
| 4200.00 | 0.136 | 0.046 | 41 | 815 | 1751 | 3718 | 757 | 1690 | 3725 | 0.027 | 96.90 |
| 4300.00 | 0.134 | 0.046 | 42 | 885 | 1899 | 4028 | 822 | 1834 | 4038 | 0.027 | 99.40 |
| 4355.00 | 0.133 | 0.046 | 42 | 922 | 1977 | 4192 | 857 | 1910 | 4204 | 0.027 | 100.69 |

# DISCHARGE = 200,000 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

OROVILLE DAM

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| ft³/s | ft | ft | | ft |
| 200000 | 16.48106 | 0.0010000 | 0.013764 | 901.0003 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 16.4811 | 67.920 | 16.399 | 899.795 | 0.00 | S2 | 8.165 | 33.940 | 2.96 | 0.696 | |
| 1350.000 | 810.34 | 0.0282 | 16.2757 | 68.777 | 15.372 | 899.193 | 0.00 | S2 | 12.344 | 33.746 | 3.00 | 1.016 | |
| 1400.000 | 807.99 | 0.0470 | 16.0655 | 69.678 | 15.138 | 898.567 | 0.00 | S2 | 10.435 | 33.756 | 3.06 | 1.318 | |
| 1500.000 | 802.32 | 0.0567 | 15.6790 | 71.395 | 15.654 | 897.240 | 0.00 | S2 | 9.817 | 33.922 | 3.18 | 1.872 | |
| 1600.000 | 796.66 | 0.0567 | 15.2450 | 73.428 | 15.221 | 895.805 | 0.00 | S2 | 9.817 | 33.933 | 3.32 | 2.392 | |
| 1700.000 | 790.99 | 0.0567 | 14.8622 | 75.319 | 14.838 | 894.251 | 0.00 | S2 | 9.817 | 33.948 | 3.45 | 2.887 | |
| 1800.000 | 785.33 | 0.0567 | 14.5223 | 77.081 | 14.499 | 892.579 | 0.00 | S2 | 9.817 | 33.966 | 3.58 | 3.362 | |
| 1900.000 | 779.66 | 0.0567 | 14.2187 | 78.727 | 14.196 | 890.793 | 0.00 | S2 | 9.817 | 33.987 | 3.70 | 3.820 | |
| 2000.000 | 773.99 | 0.0567 | 13.9462 | 80.266 | 13.924 | 888.894 | 0.00 | S2 | 9.817 | 34.011 | 3.81 | 4.265 | |
| 2100.000 | 768.33 | 0.0567 | 13.7004 | 81.705 | 13.678 | 886.887 | 0.00 | S2 | 9.817 | 34.038 | 3.92 | 4.698 | |
| 2200.000 | 762.66 | 0.0567 | 13.4781 | 83.053 | 13.457 | 884.773 | 0.00 | S2 | 9.817 | 34.068 | 4.02 | 5.120 | |
| 2300.000 | 757.00 | 0.0566 | 13.2766 | 84.314 | 13.255 | 882.558 | 0.00 | S2 | 9.819 | 34.100 | 4.12 | 5.533 | |
| 2400.000 | 749.76 | 0.0724 | 12.9494 | 86.445 | 11.966 | 880.201 | 0.00 | S2 | 9.064 | 34.032 | 4.27 | 5.938 | |
| 2500.000 | 739.39 | 0.1037 | 12.5473 | 89.215 | 11.497 | 877.627 | 0.00 | S2 | 8.070 | 34.112 | 4.50 | 6.334 | |
| 2600.000 | 725.88 | 0.1351 | 12.0600 | 92.820 | 10.929 | 874.747 | 0.00 | S2 | 7.415 | 34.215 | 4.80 | 6.723 | |
| 2700.000 | 709.23 | 0.1665 | 11.5301 | 97.085 | 10.310 | 871.457 | 0.00 | S2 | 6.941 | 34.343 | 5.16 | 7.105 | |
| 2800.000 | 689.44 | 0.1979 | 10.9909 | 101.848 | 9.674 | 867.647 | 0.00 | S2 | 6.575 | 34.494 | 5.58 | 7.481 | |
| 2900.000 | 666.52 | 0.2292 | 10.4659 | 106.957 | 9.052 | 863.203 | 0.00 | S2 | 6.284 | 34.689 | 6.06 | 7.850 | |
| 3000.000 | 642.03 | 0.2450 | 10.0316 | 111.587 | 9.744 | 858.080 | 0.00 | S2 | 6.158 | 34.966 | 6.51 | 8.214 | |
| 3100.000 | 617.53 | 0.2450 | 9.6509 | 115.989 | 9.374 | 852.284 | 0.00 | S2 | 6.158 | 35.095 | 6.93 | 8.570 | |
| 3200.000 | 593.04 | 0.2450 | 9.3343 | 119.924 | 9.066 | 845.819 | 0.00 | S2 | 6.158 | 35.230 | 7.33 | 8.921 | |
| 3300.000 | 568.54 | 0.2450 | 9.0495 | 123.698 | 8.790 | 838.686 | 0.00 | S2 | 6.158 | 35.322 | 7.71 | 9.049 | |
| 3400.000 | 544.05 | 0.2450 | 8.7700 | 127.639 | 8.518 | 830.838 | 0.00 | S2 | 6.158 | 35.322 | 8.08 | 8.770 | |
| 3500.000 | 519.55 | 0.2450 | 8.5263 | 131.287 | 8.282 | 822.241 | 0.00 | S2 | 6.158 | 35.321 | 8.43 | 8.526 | |
| 3600.000 | 495.06 | 0.2450 | 8.3121 | 134.671 | 8.073 | 812.908 | 0.00 | S2 | 6.158 | 35.320 | 8.76 | 8.312 | |
| 3700.000 | 470.56 | 0.2450 | 8.1226 | 137.813 | 7.889 | 802.854 | 0.00 | S2 | 6.158 | 35.319 | 9.07 | 8.123 | |
| 3800.000 | 446.07 | 0.2450 | 7.9540 | 140.735 | 7.726 | 792.097 | 0.00 | S2 | 6.158 | 35.318 | 9.36 | 7.954 | |
| 3900.000 | 421.57 | 0.2450 | 7.8032 | 143.454 | 7.579 | 780.656 | 0.00 | S2 | 6.158 | 35.316 | 9.63 | 7.803 | |
| 4000.000 | 397.08 | 0.2450 | 7.6678 | 145.987 | 7.448 | 768.551 | 0.00 | S2 | 6.158 | 35.314 | 9.89 | 7.668 | |
| 4100.000 | 372.58 | 0.2450 | 7.5458 | 148.347 | 7.329 | 755.805 | 0.00 | S2 | 6.158 | 35.312 | 10.13 | 7.546 | |
| 4200.000 | 348.09 | 0.2450 | 7.4355 | 150.549 | 7.222 | 742.440 | 0.00 | S2 | 6.158 | 35.323 | 10.35 | 7.435 | |
| 4300.000 | 323.59 | 0.2450 | 7.3354 | 152.602 | 7.125 | 728.480 | 0.00 | S2 | 6.158 | 35.323 | 10.57 | 7.335 | |
| 4355.000 | 310.12 | 0.2450 | 7.2843 | 153.672 | 7.075 | 720.557 | 0.00 | S2 | 6.158 | 35.323 | 10.68 | 7.284 | |

## COMPUTED FLOW PROFILE - CAVITATION PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | Initial Depth | Rugosity (ft or m) | n |
|---|---|---|---|
| ft³/s | ft | ft | |
| 200000 | 16.48105702 | 0.001 | 0.013764 |

| | | | | DAMAGE POTENTIAL | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Circular Arc | | | 90° Offset | | | | |
| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | 1/4-in (5-mm) | 1/2-in (12.5-mm) | 1-in (25-mm) | 1/4-in (5-mm) | 1/2-in (12.5-mm) | 1-in (25-mm) | Turbulence Intensity | Stream Power |
| ft | | | n : 1 | | | | | | | | kW/m² |
| 1300.00 | 0.683 | 0.037 | 4 | 0 | 1 | 2 | 0 | 2 | 6 | 0.032 | 12.16 |
| 1350.00 | 0.652 | 0.037 | 5 | 0 | 1 | 3 | 0 | 2 | 5 | 0.031 | 12.62 |
| 1400.00 | 0.632 | 0.037 | 5 | 0 | 1 | 3 | 0 | 2 | 5 | 0.030 | 13.12 |
| 1500.00 | 0.609 | 0.037 | 5 | 0 | 1 | 3 | 0 | 2 | 5 | 0.029 | 14.11 |
| 1600.00 | 0.570 | 0.037 | 6 | 0 | 1 | 4 | 0 | 2 | 6 | 0.028 | 15.35 |
| 1700.00 | 0.538 | 0.037 | 6 | 0 | 2 | 6 | 1 | 3 | 8 | 0.028 | 16.56 |
| 1800.00 | 0.510 | 0.037 | 7 | 1 | 3 | 7 | 1 | 3 | 9 | 0.027 | 17.75 |
| 1900.00 | 0.485 | 0.038 | 7 | 1 | 3 | 9 | 1 | 4 | 11 | 0.027 | 18.92 |
| 2000.00 | 0.464 | 0.038 | 7 | 1 | 4 | 11 | 1 | 5 | 13 | 0.027 | 20.05 |
| 2100.00 | 0.446 | 0.038 | 8 | 2 | 5 | 14 | 2 | 6 | 15 | 0.026 | 21.15 |
| 2200.00 | 0.429 | 0.038 | 8 | 2 | 6 | 16 | 2 | 6 | 17 | 0.026 | 22.22 |
| 2300.00 | 0.415 | 0.038 | 9 | 3 | 7 | 19 | 2 | 7 | 20 | 0.026 | 23.25 |
| 2400.00 | 0.383 | 0.038 | 10 | 4 | 11 | 27 | 4 | 11 | 27 | 0.026 | 25.07 |
| 2500.00 | 0.356 | 0.038 | 11 | 6 | 16 | 39 | 5 | 15 | 38 | 0.026 | 27.58 |
| 2600.00 | 0.325 | 0.038 | 12 | 10 | 26 | 60 | 9 | 23 | 57 | 0.025 | 31.09 |
| 2700.00 | 0.293 | 0.039 | 14 | 18 | 43 | 98 | 15 | 38 | 91 | 0.025 | 35.63 |
| 2800.00 | 0.262 | 0.039 | 16 | 31 | 73 | 165 | 25 | 63 | 148 | 0.025 | 41.21 |
| 2900.00 | 0.234 | 0.039 | 19 | 54 | 124 | 276 | 43 | 105 | 243 | 0.025 | 47.83 |
| 3000.00 | 0.219 | 0.040 | 21 | 76 | 172 | 380 | 59 | 142 | 327 | 0.025 | 54.43 |
| 3100.00 | 0.201 | 0.040 | 24 | 114 | 256 | 561 | 88 | 207 | 473 | 0.025 | 61.26 |
| 3200.00 | 0.186 | 0.040 | 26 | 162 | 360 | 781 | 123 | 286 | 648 | 0.025 | 67.84 |
| 3300.00 | 0.174 | 0.040 | 29 | 224 | 494 | 1067 | 168 | 389 | 877 | 0.025 | 74.60 |
| 3400.00 | 0.162 | 0.040 | 32 | 314 | 687 | 1476 | 237 | 543 | 1216 | 0.025 | 82.13 |
| 3500.00 | 0.152 | 0.041 | 35 | 424 | 922 | 1973 | 321 | 730 | 1629 | 0.025 | 89.55 |
| 3600.00 | 0.144 | 0.041 | 37 | 555 | 1201 | 2561 | 422 | 954 | 2119 | 0.025 | 96.83 |
| 3700.00 | 0.137 | 0.041 | 40 | 707 | 1523 | 3240 | 539 | 1213 | 2686 | 0.025 | 103.95 |
| 3800.00 | 0.131 | 0.041 | 43 | 880 | 1889 | 4009 | 672 | 1507 | 3329 | 0.025 | 110.88 |
| 3900.00 | 0.126 | 0.041 | 46 | 1073 | 2298 | 4866 | 822 | 1836 | 4047 | 0.025 | 117.61 |
| 4000.00 | 0.121 | 0.041 | 48 | 1286 | 2747 | 5807 | 987 | 2199 | 4836 | 0.025 | 124.13 |
| 4100.00 | 0.117 | 0.041 | 51 | 1517 | 3234 | 6826 | 1166 | 2592 | 5691 | 0.025 | 130.42 |
| 4200.00 | 0.113 | 0.041 | 53 | 1764 | 3754 | 7915 | 1358 | 3013 | 6606 | 0.025 | 136.48 |
| 4300.00 | 0.110 | 0.042 | 55 | 2026 | 4306 | 9067 | 1562 | 3459 | 7575 | 0.025 | 142.30 |
| 4355.00 | 0.108 | 0.042 | 56 | 2176 | 4621 | 9724 | 1679 | 3714 | 8128 | 0.025 | 145.41 |

# DISCHARGE = 290,000 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| ft³/s | ft | ft | | ft |
| 296000 | 22.87066 | 0.0010000 | 0.014079 | 917 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 22.8707 | 72.438 | 22.757 | 915.995 | 0.00 | S2 | 10.547 | 44.081 | 2.68 | 0.687 | |
| 1350.000 | 810.34 | 0.0282 | 22.5189 | 73.570 | 21.090 | 915.492 | 0.00 | S2 | 16.017 | 43.773 | 2.72 | 1.003 | |
| 1400.000 | 807.99 | 0.0470 | 22.2393 | 74.495 | 20.776 | 914.968 | 0.00 | S2 | 13.513 | 43.785 | 2.77 | 1.301 | |
| 1500.000 | 802.32 | 0.0567 | 21.7954 | 76.012 | 21.761 | 913.867 | 0.00 | S2 | 12.704 | 44.038 | 2.87 | 1.848 | |
| 1600.000 | 796.66 | 0.0567 | 21.2105 | 78.108 | 21.177 | 912.686 | 0.00 | S2 | 12.704 | 44.042 | 2.99 | 2.361 | |
| 1700.000 | 790.99 | 0.0567 | 20.6885 | 80.079 | 20.655 | 911.411 | 0.00 | S2 | 12.704 | 44.048 | 3.11 | 2.850 | |
| 1800.000 | 785.33 | 0.0567 | 20.2193 | 81.937 | 20.187 | 910.044 | 0.00 | S2 | 12.704 | 44.073 | 3.22 | 3.319 | |
| 1900.000 | 779.66 | 0.0567 | 19.7951 | 83.693 | 19.763 | 908.584 | 0.00 | S2 | 12.704 | 44.086 | 3.32 | 3.772 | |
| 2000.000 | 773.99 | 0.0567 | 19.4097 | 85.355 | 19.379 | 907.034 | 0.00 | S2 | 12.704 | 44.102 | 3.42 | 4.212 | |
| 2100.000 | 768.33 | 0.0567 | 19.0579 | 86.930 | 19.027 | 905.394 | 0.00 | S2 | 12.704 | 44.120 | 3.52 | 4.639 | |
| 2200.000 | 762.66 | 0.0567 | 18.7357 | 88.426 | 18.706 | 903.666 | 0.00 | S2 | 12.704 | 44.140 | 3.62 | 5.056 | |
| 2300.000 | 757.00 | 0.0566 | 18.4399 | 89.844 | 18.410 | 901.852 | 0.00 | S2 | 12.707 | 44.161 | 3.71 | 5.464 | |
| 2400.000 | 749.76 | 0.0724 | 17.9465 | 92.314 | 16.415 | 899.915 | 0.00 | S2 | 11.720 | 44.002 | 3.85 | 5.863 | |
| 2500.000 | 739.39 | 0.1037 | 17.3971 | 95.230 | 15.769 | 897.797 | 0.00 | S2 | 10.423 | 44.075 | 4.05 | 6.255 | |
| 2600.000 | 725.88 | 0.1351 | 16.7357 | 98.993 | 14.995 | 895.437 | 0.00 | S2 | 9.570 | 44.170 | 4.31 | 6.639 | |
| 2700.000 | 709.23 | 0.1665 | 16.0143 | 103.452 | 14.150 | 892.755 | 0.00 | S2 | 8.952 | 44.290 | 4.62 | 7.016 | |
| 2800.000 | 689.44 | 0.1979 | 15.2745 | 108.463 | 13.277 | 889.662 | 0.00 | S2 | 8.477 | 44.432 | 4.98 | 7.387 | |
| 2900.000 | 666.52 | 0.2292 | 14.5466 | 113.890 | 12.415 | 886.068 | 0.00 | S2 | 8.099 | 44.598 | 5.39 | 7.752 | |
| 3000.000 | 642.03 | 0.2450 | 13.9515 | 118.748 | 13.551 | 881.936 | 0.00 | S2 | 7.935 | 44.934 | 5.78 | 8.111 | |
| 3100.000 | 617.53 | 0.2450 | 13.4028 | 123.609 | 13.018 | 877.263 | 0.00 | S2 | 7.935 | 45.022 | 6.16 | 8.463 | |
| 3200.000 | 593.04 | 0.2450 | 12.9376 | 128.054 | 12.566 | 872.029 | 0.00 | S2 | 7.935 | 45.115 | 6.51 | 8.809 | |
| 3300.000 | 568.54 | 0.2450 | 12.5386 | 132.129 | 12.179 | 866.243 | 0.00 | S2 | 7.935 | 45.214 | 6.85 | 9.149 | |
| 3400.000 | 544.05 | 0.2450 | 12.1929 | 135.875 | 11.843 | 859.915 | 0.00 | S2 | 7.935 | 45.318 | 7.17 | 9.485 | |
| 3500.000 | 519.55 | 0.2450 | 11.8910 | 139.325 | 11.550 | 853.058 | 0.00 | S2 | 7.935 | 45.426 | 7.47 | 9.816 | |
| 3600.000 | 495.06 | 0.2450 | 11.6255 | 142.506 | 11.292 | 845.689 | 0.00 | S2 | 7.935 | 45.538 | 7.75 | 10.142 | |
| 3700.000 | 470.56 | 0.2450 | 11.3907 | 145.444 | 11.064 | 837.827 | 0.00 | S2 | 7.935 | 45.654 | 8.02 | 10.464 | |
| 3800.000 | 446.07 | 0.2450 | 11.1819 | 148.161 | 10.861 | 829.487 | 0.00 | S2 | 7.935 | 45.774 | 8.28 | 10.783 | |
| 3900.000 | 421.57 | 0.2450 | 10.9860 | 150.802 | 10.671 | 820.677 | 0.00 | S2 | 7.935 | 45.871 | 8.53 | 10.986 | |
| 4000.000 | 397.08 | 0.2450 | 10.7748 | 153.758 | 10.465 | 811.354 | 0.00 | S2 | 7.935 | 45.870 | 8.78 | 10.775 | |
| 4100.000 | 372.58 | 0.2450 | 10.5825 | 156.553 | 10.279 | 801.486 | 0.00 | S2 | 7.935 | 45.869 | 9.03 | 10.582 | |
| 4200.000 | 348.09 | 0.2450 | 10.4067 | 159.197 | 10.108 | 791.082 | 0.00 | S2 | 7.935 | 45.868 | 9.25 | 10.407 | |
| 4300.000 | 323.59 | 0.2450 | 10.2456 | 161.701 | 9.951 | 780.153 | 0.00 | S2 | 7.935 | 45.866 | 9.47 | 10.246 | |
| 4355.000 | 310.12 | 0.2450 | 10.1626 | 163.021 | 9.871 | 773.922 | 0.00 | S2 | 7.935 | 45.866 | 9.59 | 10.163 | |

## COMPUTED FLOW PROFILE - CAVITATION PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | Initial Depth | Rugosity (ft or m) | n |
|---|---|---|---|
| ft³/s | ft | ft | |
| 296000 | 22.87 | 0.001 | 0.014079 |

| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | DAMAGE POTENTIAL Circular Arc 1/4-in (5-mm) | Circular Arc 1/2-in (12.5-mm) | Circular Arc 1-in (25-mm) | 90° Offset 1/4-in (5-mm) | 90° Offset 1/2-in (12.5-mm) | 90° Offset 1-in (25-mm) | Turbulence Intensity | Stream Power |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | | | n : 1 | | | | | | | | kW/m² |
| 1300.00 | 0.678 | 0.035 | 5 | 0 | 1 | 3 | 0 | 2 | 6 | 0.032 | 14.96 |
| 1350.00 | 0.638 | 0.035 | 5 | 0 | 1 | 3 | 1 | 2 | 6 | 0.031 | 15.65 |
| 1400.00 | 0.619 | 0.035 | 5 | 0 | 1 | 4 | 0 | 2 | 6 | 0.030 | 16.24 |
| 1500.00 | 0.605 | 0.035 | 5 | 0 | 1 | 4 | 0 | 2 | 6 | 0.029 | 17.23 |
| 1600.00 | 0.567 | 0.036 | 6 | 0 | 2 | 5 | 1 | 2 | 7 | 0.028 | 18.66 |
| 1700.00 | 0.534 | 0.036 | 6 | 1 | 2 | 6 | 1 | 3 | 8 | 0.028 | 20.08 |
| 1800.00 | 0.506 | 0.036 | 7 | 1 | 3 | 8 | 1 | 4 | 10 | 0.027 | 21.48 |
| 1900.00 | 0.481 | 0.036 | 7 | 1 | 4 | 10 | 1 | 4 | 12 | 0.027 | 22.87 |
| 2000.00 | 0.459 | 0.036 | 8 | 2 | 5 | 13 | 2 | 5 | 14 | 0.027 | 24.23 |
| 2100.00 | 0.439 | 0.036 | 8 | 2 | 6 | 16 | 2 | 6 | 17 | 0.026 | 25.58 |
| 2200.00 | 0.422 | 0.036 | 8 | 3 | 7 | 19 | 2 | 7 | 20 | 0.026 | 26.90 |
| 2300.00 | 0.406 | 0.036 | 9 | 3 | 9 | 22 | 3 | 9 | 23 | 0.026 | 28.20 |
| 2400.00 | 0.370 | 0.036 | 10 | 5 | 14 | 35 | 5 | 13 | 34 | 0.026 | 30.56 |
| 2500.00 | 0.343 | 0.036 | 11 | 8 | 21 | 49 | 7 | 19 | 48 | 0.026 | 33.52 |
| 2600.00 | 0.312 | 0.037 | 13 | 13 | 33 | 76 | 11 | 30 | 72 | 0.026 | 37.62 |
| 2700.00 | 0.281 | 0.037 | 15 | 23 | 55 | 125 | 19 | 48 | 115 | 0.025 | 42.91 |
| 2800.00 | 0.251 | 0.037 | 17 | 41 | 94 | 211 | 33 | 80 | 188 | 0.025 | 49.43 |
| 2900.00 | 0.223 | 0.037 | 20 | 72 | 163 | 359 | 57 | 136 | 313 | 0.025 | 57.22 |
| 3000.00 | 0.210 | 0.038 | 22 | 96 | 216 | 473 | 74 | 176 | 403 | 0.025 | 64.88 |
| 3100.00 | 0.192 | 0.038 | 25 | 148 | 329 | 715 | 113 | 263 | 598 | 0.025 | 73.21 |
| 3200.00 | 0.177 | 0.038 | 28 | 215 | 474 | 1025 | 161 | 373 | 841 | 0.025 | 81.45 |
| 3300.00 | 0.165 | 0.038 | 31 | 299 | 654 | 1406 | 221 | 506 | 1135 | 0.025 | 89.54 |
| 3400.00 | 0.155 | 0.038 | 34 | 399 | 868 | 1860 | 290 | 662 | 1479 | 0.025 | 97.44 |
| 3500.00 | 0.146 | 0.038 | 37 | 516 | 1117 | 2386 | 370 | 840 | 1870 | 0.025 | 105.13 |
| 3600.00 | 0.139 | 0.039 | 39 | 649 | 1400 | 2981 | 460 | 1039 | 2306 | 0.025 | 112.58 |
| 3700.00 | 0.133 | 0.039 | 42 | 797 | 1715 | 3642 | 558 | 1256 | 2782 | 0.025 | 119.78 |
| 3800.00 | 0.127 | 0.039 | 45 | 959 | 2058 | 4363 | 664 | 1490 | 3293 | 0.024 | 126.71 |
| 3900.00 | 0.122 | 0.039 | 47 | 1147 | 2454 | 5194 | 788 | 1762 | 3887 | 0.024 | 133.70 |
| 4000.00 | 0.117 | 0.039 | 50 | 1408 | 3005 | 6348 | 969 | 2162 | 4759 | 0.024 | 141.84 |
| 4100.00 | 0.112 | 0.039 | 53 | 1701 | 3623 | 7641 | 1175 | 2612 | 5738 | 0.025 | 149.83 |
| 4200.00 | 0.108 | 0.039 | 56 | 2028 | 4310 | 9076 | 1403 | 3112 | 6824 | 0.025 | 157.68 |
| 4300.00 | 0.105 | 0.039 | 59 | 2387 | 5064 | 10649 | 1654 | 3662 | 8018 | 0.025 | 165.37 |
| 4355.00 | 0.103 | 0.039 | 61 | 2598 | 5506 | 11573 | 1802 | 3985 | 8719 | 0.025 | 169.52 |

# DISCHARGE = 300,000 cfs

## COMPUTED FLOW PROFILE - HYDRAULIC PROPERTIES

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | $Y_o$ | Rugosity | n | EGL at Crest |
|---|---|---|---|---|
| ft³/s | ft | ft | | ft |
| 300000 | 26.53734 | 0.0010000 | 0.014223 | 901.0019 |

| Station | Invert Elevation | Slope | Depth | Velocity | Piez | Energy Grade Line | Q Air/ Q Water | Profile | Normal Depth | Critical Depth | Froude Number | Thickness Boundary Layer | Roll Wave Check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | ft | | ft | ft/s | ft | ft | | | ft | ft | | ft | |
| 1300.000 | 811.75 | 0.1000 | 26.5373 | 63.273 | 26.406 | 900.326 | 0.00 | S2 | 10.640 | 44.480 | 2.17 | 0.706 | |
| 1350.000 | 810.34 | 0.0282 | 25.9620 | 64.675 | 24.686 | 899.988 | 0.00 | S2 | 16.162 | 44.166 | 2.23 | 1.030 | |
| 1400.000 | 807.99 | 0.0470 | 25.4739 | 65.915 | 24.156 | 899.629 | 0.00 | S2 | 13.634 | 44.178 | 2.29 | 1.335 | |
| 1500.000 | 802.32 | 0.0567 | 24.6855 | 68.020 | 24.646 | 898.855 | 0.00 | S2 | 12.817 | 44.437 | 2.42 | 1.895 | |
| 1600.000 | 796.66 | 0.0567 | 23.7619 | 70.663 | 23.724 | 897.998 | 0.00 | S2 | 12.817 | 44.443 | 2.56 | 2.419 | |
| 1700.000 | 790.99 | 0.0567 | 22.9679 | 73.106 | 22.931 | 897.047 | 0.00 | S2 | 12.817 | 44.450 | 2.69 | 2.916 | |
| 1800.000 | 785.33 | 0.0567 | 22.2754 | 75.379 | 22.240 | 896.001 | 0.00 | S2 | 12.817 | 44.458 | 2.82 | 3.393 | |
| 1900.000 | 779.66 | 0.0567 | 21.6644 | 77.505 | 21.630 | 894.862 | 0.00 | S2 | 12.817 | 44.468 | 2.94 | 3.853 | |
| 2000.000 | 773.99 | 0.0567 | 21.1205 | 79.501 | 21.087 | 893.630 | 0.00 | S2 | 12.817 | 44.479 | 3.06 | 4.299 | |
| 2100.000 | 768.33 | 0.0567 | 20.6327 | 81.380 | 20.600 | 892.307 | 0.00 | S2 | 12.817 | 44.507 | 3.17 | 4.732 | |
| 2200.000 | 762.66 | 0.0567 | 20.1925 | 83.155 | 20.160 | 890.893 | 0.00 | S2 | 12.817 | 44.525 | 3.27 | 5.154 | |
| 2300.000 | 757.00 | 0.0566 | 19.7934 | 84.831 | 19.762 | 889.390 | 0.00 | S2 | 12.820 | 44.546 | 3.38 | 5.567 | |
| 2400.000 | 749.76 | 0.0724 | 19.1678 | 87.600 | 17.691 | 887.765 | 0.00 | S2 | 11.824 | 44.382 | 3.54 | 5.970 | |
| 2500.000 | 739.39 | 0.1037 | 18.4832 | 90.845 | 16.902 | 885.960 | 0.00 | S2 | 10.515 | 44.454 | 3.75 | 6.365 | |
| 2600.000 | 725.88 | 0.1351 | 17.6825 | 94.958 | 15.979 | 883.918 | 0.00 | S2 | 9.654 | 44.549 | 4.02 | 6.753 | |
| 2700.000 | 709.23 | 0.1665 | 16.8301 | 99.768 | 14.994 | 881.559 | 0.00 | S2 | 9.031 | 44.669 | 4.34 | 7.133 | |
| 2800.000 | 689.44 | 0.1979 | 15.9737 | 105.116 | 13.995 | 878.799 | 0.00 | S2 | 8.552 | 44.813 | 4.72 | 7.506 | |
| 2900.000 | 666.52 | 0.2292 | 15.1458 | 110.862 | 13.025 | 875.547 | 0.00 | S2 | 8.170 | 44.981 | 5.14 | 7.873 | |
| 3000.000 | 642.03 | 0.2450 | 14.4757 | 115.994 | 14.060 | 871.767 | 0.00 | S2 | 8.004 | 45.304 | 5.54 | 8.233 | |
| 3100.000 | 617.53 | 0.2450 | 13.8655 | 121.100 | 13.467 | 867.451 | 0.00 | S2 | 8.004 | 45.404 | 5.93 | 8.587 | |
| 3200.000 | 593.04 | 0.2450 | 13.3518 | 125.759 | 12.968 | 862.579 | 0.00 | S2 | 8.004 | 45.498 | 6.29 | 8.934 | |
| 3300.000 | 568.54 | 0.2450 | 12.9136 | 130.026 | 12.543 | 857.157 | 0.00 | S2 | 8.004 | 45.597 | 6.63 | 9.276 | |
| 3400.000 | 544.05 | 0.2450 | 12.5357 | 133.945 | 12.176 | 851.195 | 0.00 | S2 | 8.004 | 45.700 | 6.96 | 9.612 | |
| 3500.000 | 519.55 | 0.2450 | 12.2070 | 137.553 | 11.856 | 844.704 | 0.00 | S2 | 8.004 | 45.809 | 7.27 | 9.944 | |
| 3600.000 | 495.06 | 0.2450 | 11.9188 | 140.879 | 11.577 | 837.698 | 0.00 | S2 | 8.004 | 45.921 | 7.56 | 10.271 | |
| 3700.000 | 470.56 | 0.2450 | 11.6645 | 143.949 | 11.330 | 830.192 | 0.00 | S2 | 8.004 | 46.038 | 7.84 | 10.594 | |
| 3800.000 | 446.07 | 0.2450 | 11.4389 | 146.789 | 11.110 | 822.209 | 0.00 | S2 | 8.004 | 46.158 | 8.11 | 10.914 | |
| 3900.000 | 421.57 | 0.2450 | 11.2377 | 149.416 | 10.915 | 813.765 | 0.00 | S2 | 8.004 | 46.282 | 8.36 | 11.229 | |
| 4000.000 | 397.08 | 0.2450 | 11.0131 | 152.464 | 10.697 | 804.816 | 0.00 | S2 | 8.004 | 46.283 | 8.62 | 11.013 | |
| 4100.000 | 372.58 | 0.2450 | 10.8082 | 155.354 | 10.498 | 795.321 | 0.00 | S2 | 8.004 | 46.282 | 8.86 | 10.808 | |
| 4200.000 | 348.09 | 0.2450 | 10.6212 | 158.090 | 10.316 | 785.289 | 0.00 | S2 | 8.004 | 46.281 | 9.10 | 10.621 | |
| 4300.000 | 323.59 | 0.2450 | 10.4500 | 160.679 | 10.150 | 774.728 | 0.00 | S2 | 8.004 | 46.280 | 9.32 | 10.450 | |
| 4355.000 | 310.12 | 0.2450 | 10.3619 | 162.045 | 10.064 | 768.699 | 0.00 | S2 | 8.004 | 46.279 | 9.44 | 10.362 | |

**COMPUTED FLOW PROFILE - CAVITATION PROPERTIES**

**OROVILLE DAM**

Compute Flow Profile & Cavitation Output

| Q | Initial Depth | Rugosity (ft or m) | n |
|---|---|---|---|
| $ft^3/s$ | ft | ft | |
| 300000 | 26.54 | 0.001 | 0.014223 |

| Station | Flow Sigma | Sigma of Uniform Roughness | Required Chamfer to Stop Cavitation | DAMAGE POTENTIAL: Circular Arc 1/4-in (5-mm) | Circular Arc 1/2-in (12.5-mm) | Circular Arc 1-in (25-mm) | 90° Offset 1/4-in (5-mm) | 90° Offset 1/2-in (12.5-mm) | 90° Offset 1-in (25-mm) | Turbulence Intensity | Stream Power |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ft | | | n : 1 | | | | | | | | $kW/m^2$ |
| 1300.00 | 0.948 | 0.035 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.032 | 10.11 |
| 1350.00 | 0.881 | 0.035 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.031 | 10.77 |
| 1400.00 | 0.840 | 0.035 | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 0.030 | 11.38 |
| 1500.00 | 0.796 | 0.035 | 4 | 0 | 0 | 1 | 0 | 0 | 1 | 0.029 | 12.47 |
| 1600.00 | 0.725 | 0.035 | 4 | 0 | 0 | 1 | 0 | 0 | 2 | 0.028 | 13.93 |
| 1700.00 | 0.668 | 0.035 | 5 | 0 | 0 | 2 | 0 | 1 | 2 | 0.028 | 15.38 |
| 1800.00 | 0.621 | 0.035 | 5 | 0 | 1 | 2 | 0 | 1 | 3 | 0.027 | 16.82 |
| 1900.00 | 0.581 | 0.035 | 6 | 0 | 1 | 4 | 0 | 1 | 4 | 0.027 | 18.25 |
| 2000.00 | 0.546 | 0.036 | 6 | 0 | 2 | 5 | 0 | 2 | 6 | 0.027 | 19.67 |
| 2100.00 | 0.517 | 0.036 | 6 | 1 | 2 | 7 | 1 | 2 | 7 | 0.026 | 21.07 |
| 2200.00 | 0.491 | 0.036 | 7 | 1 | 3 | 8 | 1 | 3 | 9 | 0.026 | 22.45 |
| 2300.00 | 0.468 | 0.036 | 7 | 1 | 4 | 11 | 1 | 4 | 11 | 0.026 | 23.81 |
| 2400.00 | 0.421 | 0.036 | 8 | 2 | 7 | 18 | 2 | 7 | 18 | 0.026 | 26.18 |
| 2500.00 | 0.386 | 0.036 | 10 | 4 | 11 | 27 | 3 | 10 | 27 | 0.026 | 29.16 |
| 2600.00 | 0.346 | 0.036 | 11 | 7 | 19 | 46 | 6 | 17 | 43 | 0.025 | 33.25 |
| 2700.00 | 0.307 | 0.037 | 13 | 14 | 35 | 81 | 12 | 31 | 74 | 0.025 | 38.52 |
| 2800.00 | 0.271 | 0.037 | 15 | 27 | 64 | 146 | 22 | 55 | 130 | 0.025 | 45.01 |
| 2900.00 | 0.239 | 0.037 | 18 | 51 | 118 | 262 | 40 | 98 | 228 | 0.025 | 52.78 |
| 3000.00 | 0.223 | 0.037 | 20 | 72 | 163 | 361 | 56 | 133 | 308 | 0.025 | 60.45 |
| 3100.00 | 0.202 | 0.038 | 23 | 115 | 258 | 565 | 88 | 207 | 472 | 0.025 | 68.81 |
| 3200.00 | 0.185 | 0.038 | 26 | 173 | 384 | 832 | 130 | 301 | 683 | 0.025 | 77.10 |
| 3300.00 | 0.172 | 0.038 | 29 | 247 | 542 | 1168 | 182 | 418 | 942 | 0.025 | 85.27 |
| 3400.00 | 0.160 | 0.038 | 32 | 336 | 734 | 1575 | 244 | 558 | 1251 | 0.025 | 93.28 |
| 3500.00 | 0.151 | 0.038 | 35 | 442 | 961 | 2055 | 317 | 721 | 1609 | 0.025 | 101.09 |
| 3600.00 | 0.143 | 0.038 | 38 | 565 | 1221 | 2605 | 399 | 904 | 2012 | 0.025 | 108.68 |
| 3700.00 | 0.136 | 0.039 | 41 | 703 | 1515 | 3223 | 491 | 1108 | 2458 | 0.025 | 116.02 |
| 3800.00 | 0.130 | 0.039 | 43 | 856 | 1839 | 3903 | 591 | 1329 | 2942 | 0.024 | 123.11 |
| 3900.00 | 0.125 | 0.039 | 46 | 1022 | 2190 | 4642 | 698 | 1566 | 3459 | 0.024 | 129.93 |
| 4000.00 | 0.120 | 0.039 | 49 | 1266 | 2705 | 5720 | 867 | 1937 | 4269 | 0.024 | 138.15 |
| 4100.00 | 0.115 | 0.039 | 52 | 1543 | 3290 | 6944 | 1060 | 2361 | 5191 | 0.024 | 146.28 |
| 4200.00 | 0.110 | 0.039 | 55 | 1854 | 3943 | 8311 | 1276 | 2835 | 6222 | 0.024 | 154.26 |
| 4300.00 | 0.106 | 0.039 | 58 | 2197 | 4666 | 9820 | 1516 | 3360 | 7362 | 0.025 | 162.09 |
| 4355.00 | 0.104 | 0.039 | 59 | 2400 | 5092 | 10710 | 1658 | 3670 | 8035 | 0.025 | 166.33 |

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# Appendix C

# General Site Geology, Seismicity, and Site Geological Conditions

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## 1.0 GENERAL SITE GEOLOGY

Up until the project was constructed, only general and broad descriptions of the regional geology and bedrock types to be found at the project site were available from published sources. In the Final Geologic Report [C-1] the geology of the site area is described as comprising steeply dipping metamorphic rocks of the “Bedrock Series,” ranging in age from questionable later Paleozoic to questionable Middle Jurassic. According to this report, most of the reservoir area and the foundations at Oroville Dam and Spillways involve rock units of an unnamed metavolcanic member, which is the oldest member of the “Bedrock Series.” This member is indicated as being predominantly **amphibolite** or **amphibolite schist**, though in some reports it is described as “greenstone.”

One of the earliest geologic maps of the region was published by California Division of Mines and Geology in 1962 (see Figure C-1). This map shows bedrock at the dam site and spillway areas to be undifferentiated Jura-Trias metavolcanic rocks.

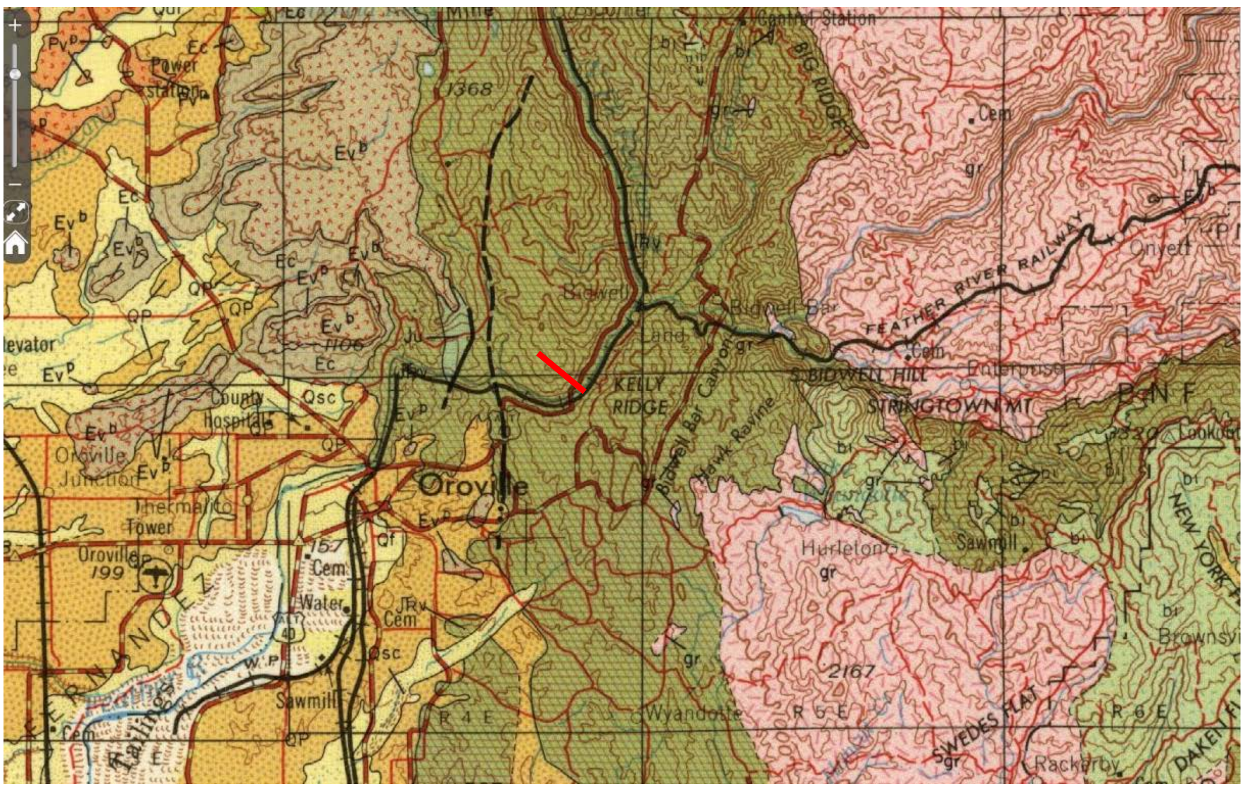



**Figure C-1: Enlarged portion of the Chico Quadrangle Geologic Map [C-2], approximate location of Oroville Dam shown by red line**

In addition to this source, it is likely that much of the background information and nomenclature on the “Bedrock Series” used in DWR reports on the dam and spillway geology had been summarized from the work of R.S. Creely, including his 1955 PhD thesis [C-3] and the Geology of the Oroville Quadrangle published in 1965 [C-4]. Creely provided petrographic descriptions that frequently used the term “schist” because of the fine-scale foliation he observed.

Subsequently, the bedrock units were named as part of the Smartville Ophiolite Complex and shown on geologic maps published by the California DWR [C-5], Figure C-2, and the California Geologic Survey [C-6], Figure C-3.

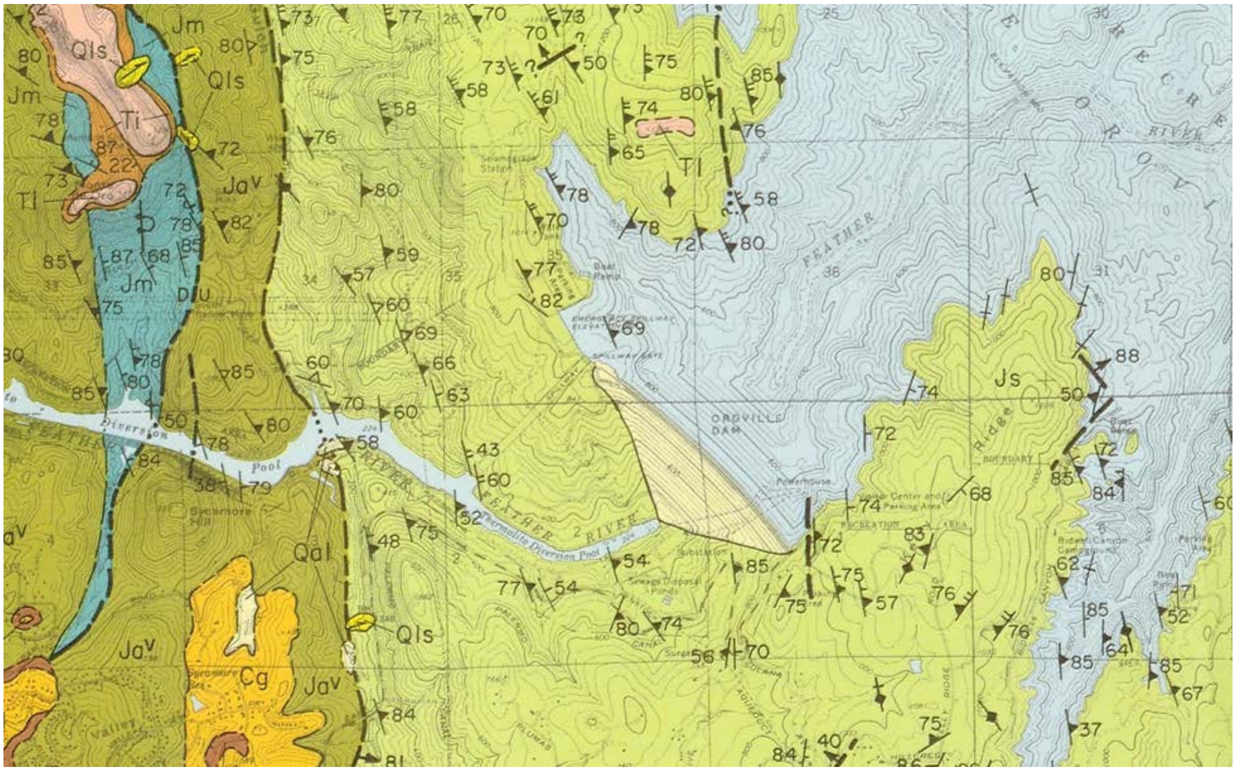



**Figure C-2: Enlarged portion of the geologic map of Cole & McJunkin (1978)**

Legend descriptions on these maps indicate the Smartville Ophiolite Complex as being comprised of dark gray to green gray, steeply-dipping, strongly foliated, metamorphosed, basaltic to diabasic volcaniclastic sediment, pillow lava, breccia, dikes and sills; gabbroic to felsic screen rocks occur within sheeted dikes; gabbroic plugs are rare. The bedrock units are dark gray to green gray in color, and are strongly foliated, dipping steeply to the northeast. This revised terminology of the bedrock units has been generally adopted by DWR since the 1980s, although the generic usage of "amphibolite" remains in common usage.

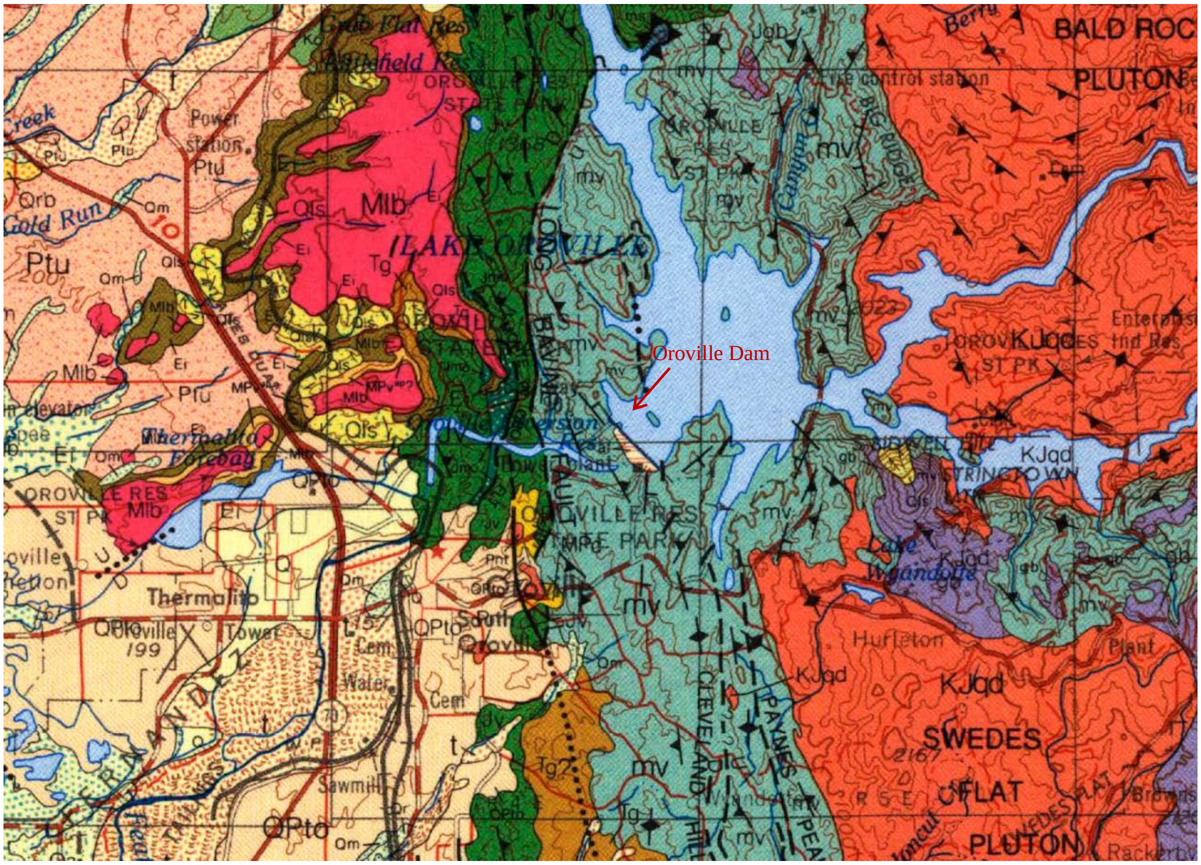



**Figure C-3: Enlarged Portion of the Chico Quadrangle Geologic Map[C-6]**

It is evident that a thick cover of saprolite (strongly weathered and decomposed bedrock) has developed in areas underlain by the Smartville Ophiolite sequence, as observed in the erosional damage resulting from the February 2017 incident. This should not have come as a total surprise, given the nature of ophiolite and amphibolite sequences that include basic and mafic igneous rock types. Such lithologies commonly include unstable mineral assemblages that readily weather and degrade in comparison with many other lithologies.

The Smartville Ophiolite Complex is described as comprising rock types that can be placed in this category, including basalt, diabase, pillow lava, and gabbro. There are some reports that the Smartville ophiolite contains large amounts of serpentine and other ultramafic rocks, which are well-known for containing the most pervasive, unstable, and low-strength mineral assemblages. However, there are no direct reports indicating presence of ultramafics, serpentinite, or spilitic sequences in the foundation bedrock at the Oroville Project.

Nevertheless, development of thick saprolitic profiles and extensive, deep weathering is commonly observed in association with ophiolites in humid temperate to subtropical environments (similar to California), especially if the ophiolites are severely deformed and fractured. In addition, the mineralogy of many ophiolite sequences, including the Smartville Complex, has been altered by hydrothermal and/or other types of alteration processes that would also contribute to breakdown and weathering upon exposure to oxidizing groundwater close to the earth's surface.

**IFT Comments**: There is reasonably common knowledge in current practice that foundations involving rock descriptions as given (i.e. amphibolite, greenstone, or ophiolite) might point toward a particular susceptibility or tendency for pronounced weathering, and that the weathered by-products could be vulnerable to erosion. The IFT concludes that, since the 1980s at least, based simply on regional geological mapping, a qualified engineering geologist should have been able to recognize this potential issue at Oroville. If this recognition had been made post-construction, it should have led to (1) questioning if such conditions had been properly recognized and managed during design and construction (e.g. having provisions for appropriate adjustments in excavation depths, anchor lengths, or other measures); and (2) questioning the erodibility of the foundations, particularly if there is knowledge that weathered materials had been left in place. It is noted however that the observations made during construction were generally consistent with what the regional geology suggests and with petrologic knowledge at that time.

## 2.0 SEISMICITY

As part of the IFT examination of factors that could have contributed to the February 2017 incident, a brief study was conducted regarding whether earthquake activity was likely to have been a significant influence. Since the failure of the service spillway occurred several decades after the project went into operation, the IFT has examined factors that may have changed or degraded over time and possibly have helped trigger the spillway chute failure scenario. This particular enquiry into seismicity is part of this broader investigation and focused on evaluation of whether any recent earthquake events could be considered potential contributory factors.

### 2.1 Seismotectonic Setting

The Oroville Project is in an area of northeastern California characterized by relatively low seismic activity in historic times. Overall, the Sierra Nevada and Central Valley move together as an independent block, the eastern margin of which is formed by faults of the Sierra Nevada Fault Zone. Two fault types offset rocks in the area: high-angle reverse faults in the Sierra Nevada Geomorphic Province and normal faults in the Sierra Nevada and Cascade Range Geomorphic Provinces. The dominant structure of the Sierra Nevada metamorphic belt and the project area is the Foothills Fault System. This series of north-northwest trending, east-dipping reverse faults was formed during the late Jurassic era, when subduction along the western continental margin resulted in the Nevadan orogeny. The Foothills Fault System, although considered relatively quiet seismically, is considered an important source given the influence of this system on the geologic structure of the project region. Seismicity on these faults has been reactivated in the late Cenozoic era [C-7].

### 2.2 Seismic Activity

An assessment was made of seismic activity recorded in the project region since the project went into operation in 1968, with a focus on activity occurring in the periods since the last major spillway flows (prior to the February 2017 incident) took place in 2006. The purpose was to determine if any earthquake events had occurred in such a timeframe that may have generated ground motions of sufficient strength or duration to have potentially damaged or weakened the spillway facilities at Oroville Dam. Strong ground motions sufficient to have caused damage would had to have been either large events at some distance from Oroville Dam or smaller events close to the site.

The most significant recorded seismicity to have affected the Oroville Project involved a series of earthquakes occurring in the summer of 1975, about 8 to 10 miles south of Lake Oroville [C-8]. The largest event was on August 1, 1975, with a recorded magnitude M= 5.7 to 5.8, which was accompanied by surface faulting [C-9]. The earthquake sequence, consisting of five foreshocks, a main shock, and numerous aftershocks, included seven events of magnitude greater than 4.6 [C-10]. Critical facilities such as dams, pumping and power plants, switchyards, pipelines, and canals were all inspected and evaluated for safety following the earthquakes.

Several investigators proposed that the 1975 series of events was a compelling example of reservoir triggered seismicity (RTS), pointing out two factors suggesting that Lake Oroville could have contributed to both the location and timing of the events. One factor is the proximity of the seismic events to the lake, and the second factor is the occurrence of the earthquake swarm following an unprecedented seasonal fluctuation in lake levels [C-8]. During the winter of 1974-1975, the lake was drawn down to its lowest level since initial filling. This exceptional drawdown and subsequent refilling was followed by the earthquake sequence of 1975. Such phenomena have been observed at other large reservoir projects and the present-day consensus of scientists, seismologists, and dam engineers is that RTS is not only plausible, but should be taken into account in design and operation of large reservoir projects [C-11].

During the last drought period, the level of Lake Oroville dropped to 26% capacity in late 2015 and slowly started recovery in 2016, reaching 96% capacity in May before being drawn down again in the summer and fall. However, the rate of refilling in the last winter period was exceptional. Last winter (2016-2017) it took only two months to fill Lake Oroville, whereas in 2015-2016 it took five months.

Therefore, it is reasonable to ask if earthquake occurrence, including possible RTS, could have contributed to the February 2017 incident. However, based on information gathered by the IFT, there are cogent facts and explanations as to why seismicity and RTS are not considered plausible contributory factors. These include:

1. No ground motions generated by earthquake events, and strong enough conceivably to cause damage to large civil structures, have been recorded by instrumentation at the project within the last 20 years.
2. Since the 1975 events, there have been no earthquakes recorded greater than M=3.9 within 30 miles of the project. Dozens of microearthquakes (<M=2.0) are recorded every year, but these are mostly not felt by humans and are not damaging to major civil structures.
3. The service spillway chute slabs and its foundations are not as vulnerable to the effects of strong ground motions as certain other facilities at the project, such as the FCO headworks, which have had no observable seismic damage.
4. A plot of earthquakes (>M=2.5) recorded in the last 20 years and within 125 miles (200 km) of the project is provided in Figure C-4. The data set includes seven events M 4.0 to M5.7 but each was located more than 100 miles from Oroville Dam. The remainder are all

<M4.0. These earthquakes occurred during the timeframe when previous historical major flows had taken place over the spillway. In other words, no new potential damage caused by seismicity had taken place in the time between previous major spillway releases and the February 2017 spillway flows.

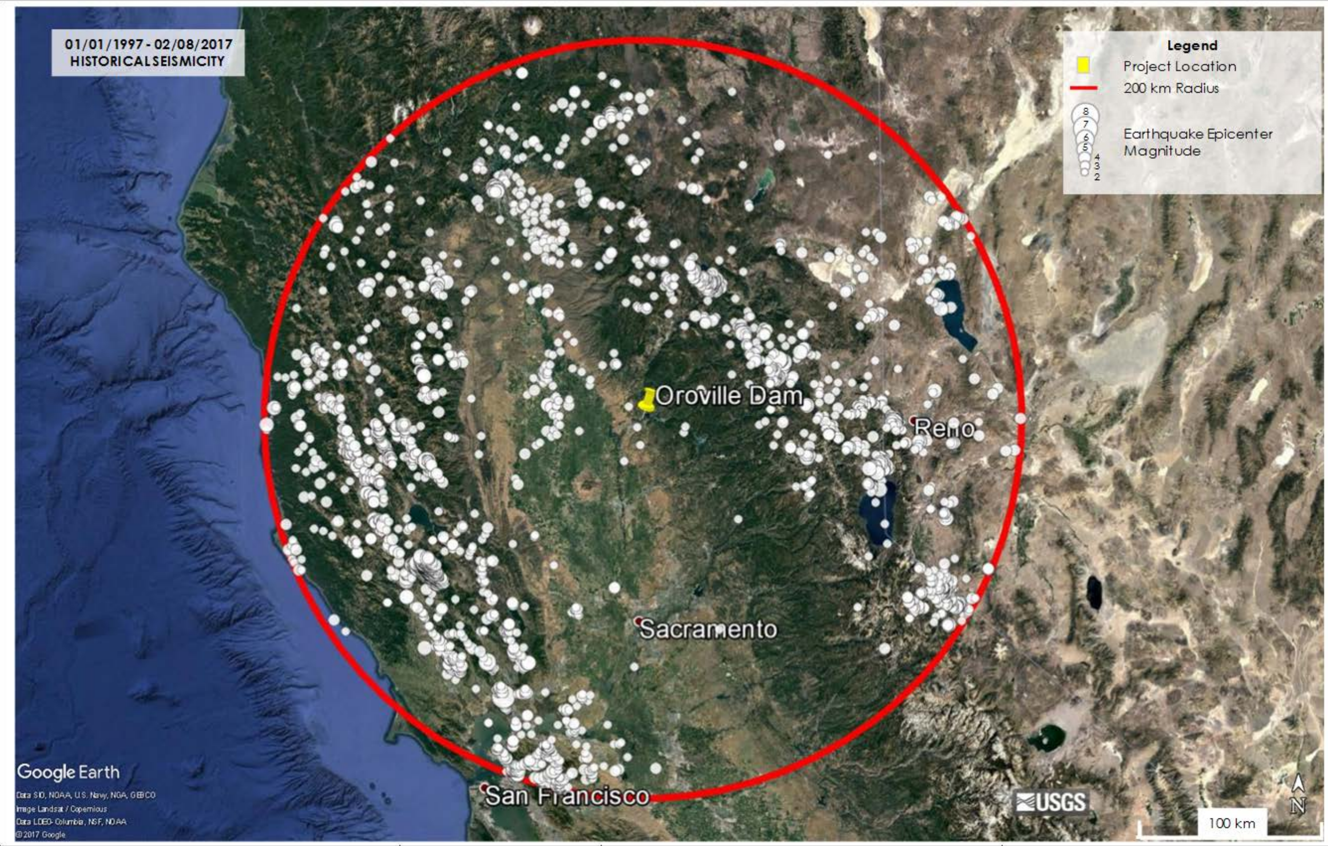



**Figure C-4: Plot of earthquakes (>M=2.5) recorded in the last 20 years and within 125 miles (200 km) of Oroville Dam. Data from USGS Search Earthquake Catalog [C-12].**

## 3.0 SITE GEOLOGICAL CONDITIONS AS DOCUMENTED (1948-2009)

The misconceptions regarding the erodibility of the bedrock along the two spillway alignments are central to the development of the Oroville incident. The remainder of this appendix traces the development of geological knowledge and geotechnical assessments of the spillway foundations from earliest investigations, through construction, and up to the point where the misconceptions were thoroughly entrenched and not seriously questioned.

### 3.1 <u>Initial Explorations (Emergency Spillway Area)</u>

The U.S. Department of the Interior, Bureau of Reclamation (Reclamation) drilled six core holes during a 1948 reconnaissance exploratory program. Their report [C-13] notes that the U.S. Army Corp of Engineers (USACE) had previously drilled two holes in the general site area, however documentation of this earlier investigation was not located by the IFT. The Reclamation report notes that there was no knowledge of any prior geological literature referring to the Oroville area "except that made before the turn of the century by the US Geological Survey," but that the regional geology had been covered in a 1938 Geological Map of California.

The Reclamation report comments on rock conditions as follows:

> "Fresh rock is exposed in the stream section. It is generally very hard, competent and watertight but contains small gouge seams, shear seams and weak zones, as well as irregular joints. Varying thicknesses of weathered and broken rock overlie the fresh rock on the hillsides."
>
> "The fresh rock is hard and resistant to normal erosion."
>
> "The site should be thoroughly tunneled to learn of the extent and subsurface condition of seams."
>
> "From the top soil to fresh rock at depth the following zones of weathered rock are generally present: (a) Zone of rock that is entirely decomposed to clay; (b) A zone of completely decomposed rock in which the weathering has not entirely destroyed the textures and structures of the rock; (c) A zone in which the rock is partly decomposed and partly fresh; (d) A zone of fresh, hard and firm rock which contains mud seams and weathered joints; and (e) A zone of fresh rock that has not been effected by the forces of weathering."
>
> "In logging the core, particular emphasis was placed on the degree of weathering and the presence of gouge seams."
>
> "In the more favorable holes the top soil is 0 to 2 feet thick; the entirely decomposed rock zone extending from 6 inches to 6 feet …; the partially decomposed rock zone extending from 11 to 40 feet …; the zone of fresh rock with weathered seams extending from 34 to 59 feet …"
>
> "On the right abutment in the vicinity of Drill Hole Nos. IF2 and 7, the effects of weathering generally extend deeper than in other areas drilled. The U. S. Engineers' Drill Hole No. IF2 was in rock with decomposed seams at a depth of 152 feet when the hole was abandoned. Drill Hole No. 7 shows … partially decomposed rock which extends to a vertical depth of 38 feet … [and] the hole reached entirely fresh rock at a vertical depth of 80 feet."

Thus, in the first preliminary explorations, the depth and extent of bedrock weathering is clearly identified and documented. However, the description provides an idealized view of a rock weathering profile, gradually increasing in quality with depth. In reality, a deep rock weathering profile is typically very complex, with occasions where much higher degrees of weathering can occur below good quality rock, and even fully surround essentially fresh rock. Rock weathering can also vary greatly over very small distances horizontally, making interpolation between boreholes quite imprecise. Later geologic reports better emphasize the great variability in rock weathering.

This 1948 report also specifically comments on the erodibility of the hillside along what is now the emergency spillway, and proposes an engineering solution:

> "Spillway for Earthfill Dam. … Drill Hole No. 5, bored at the proposed spillway gates, was logged as follows: 0 to 11 feet decomposed rock, and 11 to 23 feet fresh rock with stained joints. As shown by the numerous road cuts … decomposed rock will stand temporarily on a 1 to 1 slope ...
>
> The hard resistant rock in the proposed spillway draw will prevent spillway water from eroding back to the dam. However, the volume of material eroded and deposited in the river will be large if the spillway channel is left unlined. Therefore, deep cut-off to fresh rock at the end of the lined channel will be necessary to prevent serious erosion damage to the spillway channel and gate structure in any single flood season."

Clearly, the 1948 Reclamation investigators had a grasp of the erodibility issue, and actually proposed a solution that is now being utilized in the remediation works – a deep cutoff to fresh rock.

### 3.2 1950s Explorations

The future spillway areas were next investigated by way of 51 bulldozer trenches in 1952 and 1953, followed by five core holes near the spillway crest in 1956. The reports of these investigations were not available for review.

### 3.3 1961 Explorations (Emergency Spillway Area)

In 1961, DWR completed six seismic refraction spreads and approximately 22 diamond core holes in the general vicinity of the spillways. Results are given in two reports; a December 1961 report "Interim Report Riprap Explorations, Oroville Spillway" and a June1962 report "Interim Report of Geological Investigation."

The first report [C-14] was specifically written to summarize the potential for extracting riprap (large durable rock for placement on the upstream face of the Oroville Dam). The report notes 22 boreholes, but does not present any borehole logs. It does, however, present an interpretation of "depth to sound rock," in order to estimate the total amount of overlying material (all of which was referred to as "overburden," as distinct from quarry rock) that would have to be removed prior to establishing a quarry. Sound rock was defined as follows:

> "Sound rock will include fresh and slightly weathered rock with unstained or slightly iron-stained fractures. The rock may be slightly to moderately fractured. Material above sound rock surface will include the following:
>
> 1. All soil and decomposed rock.
> 2. All strongly and moderately weathered rock, and all slightly weathered and fresh rock that is sheared, strongly fractured, or has strongly iron-stained fractures, except zones which are shorter in drill-hole length than an overlying interval classified as sound rock."

Thus, the evaluation at the time clearly accounted for the fact that "soundness" is not only a function of the degree of weathering, but also of shearing and fracturing. The interpretation of "depth to sound rock" was carried over from this report to succeeding geological reports as discussed in later sections of this appendix.

The second report [C-15] is broader in scope, and includes consideration of foundation conditions for the spillway chute, expectations for which could be relaxed from those necessary for the production of riprap.

The service spillway alignment under consideration at the time was within the location of the current emergency spillway, and the boreholes for the spillway chute followed down a prominent topographic depression leading from the left side of the emergency spillway to the Feather River. Site conditions were summarized as follows:

> "Amphibolite is hard when slightly weathered, and very soft and crumbly when decomposed. The soil zone usually grades down into decomposed rock and the rock is generally progressively less weathered with depth. However, there may be cases where weathered zones occur below relatively fresh rock."
>
> "Depth of weathering varies from 0 to at least 76 feet, as determined from core drilling, and may extend to 95 feet beneath ground surface, as indicated by seismic exploration. Apparently, the rock along much of the lower spillway is weathered to considerable depths. Depth to weathering ranges from about 40 feet at Stations 38+00 and 50+00 to possibly 95 feet at Station 44+00. Depth of weathering appears to be shallow between Stations 30+00 and 37+00, ranging from 0 to approximately 25 feet. Except for localized areas, principally areas of rock outcrop, depth of weathering over the remainder of the spillway will generally range between 15 and 30 feet."
>
> "Because weathering is generally accentuated along structural features such as sheared or foliated zones, deeply weathered pockets or zones may exist in close proximity to outcrop areas or areas of shallow weathering."

This describes a typical deep weathering pattern in bedrock, and clearly recognizes its very irregular pattern, including the development of what would now be termed "corestones," where the weathering process has preferentially followed the rock structure, and has totally encapsulated areas of relatively unweathered rock. Thus, encountering unweathered rock at a particular depth in any borehole location is not necessarily "proof" that such conditions will persist over the general depth and area.

The report not only describes the geology, but also accurately opines on its probable behavior including erosion potential, providing what would now be called a geotechnical assessment:

> "Fresh rock will be highly resistant to scour; however, some localized zones of strongly jointed, foliated or sheared rock will probably be encountered within the fresh rock at spillway invert, and plucking from these zones can be expected. Rock bolts and/or shallow consolidation grouting could be used to stabilize such zones. Weathered rock will of course be subject to relatively accelerated erosion; where this is critical, the rock should be protected."

"Depth to sound rock," defined similarly to that in the first report, was shown in a longitudinal section along what is now an alignment within the emergency spillway discharge channel. The proposed invert of the spillway on the profile was shown to be excavated down into "sound rock," or at the top of "sound rock" for its entirety, except for a small portion adjacent to the Feather River. A portion of the profile is shown below in Figure C-5.

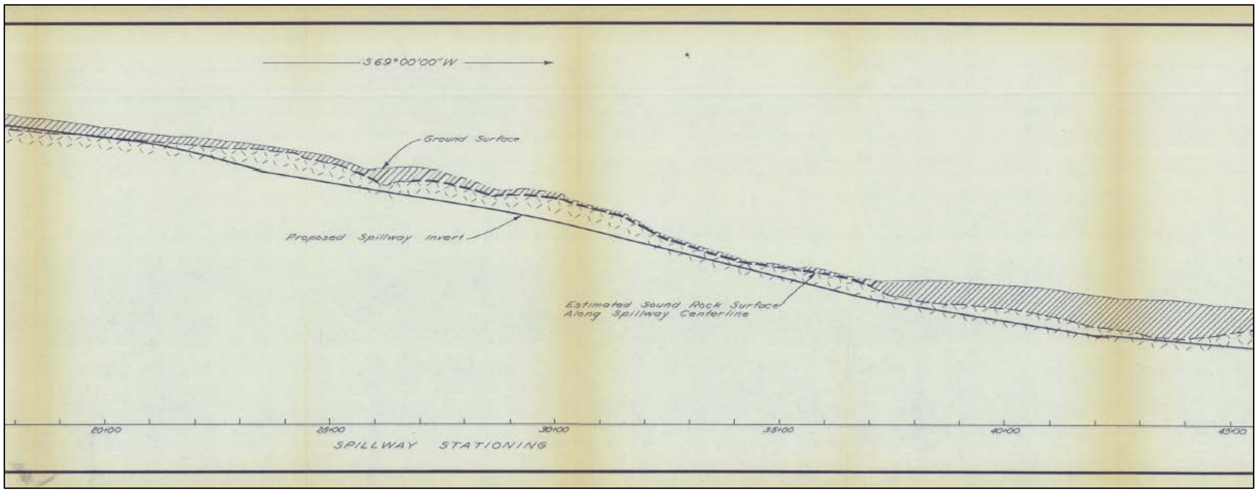



**Figure C-5: Portion of a longitudinal section along original spillway alignment, based on 1961 investigations. Now downstream from current emergency spillway.**

This foundation rock had been thought of as acceptable for riprap, and the IFT believes that this is the basis of not having a concrete liner in the original design, as noted later in this appendix.

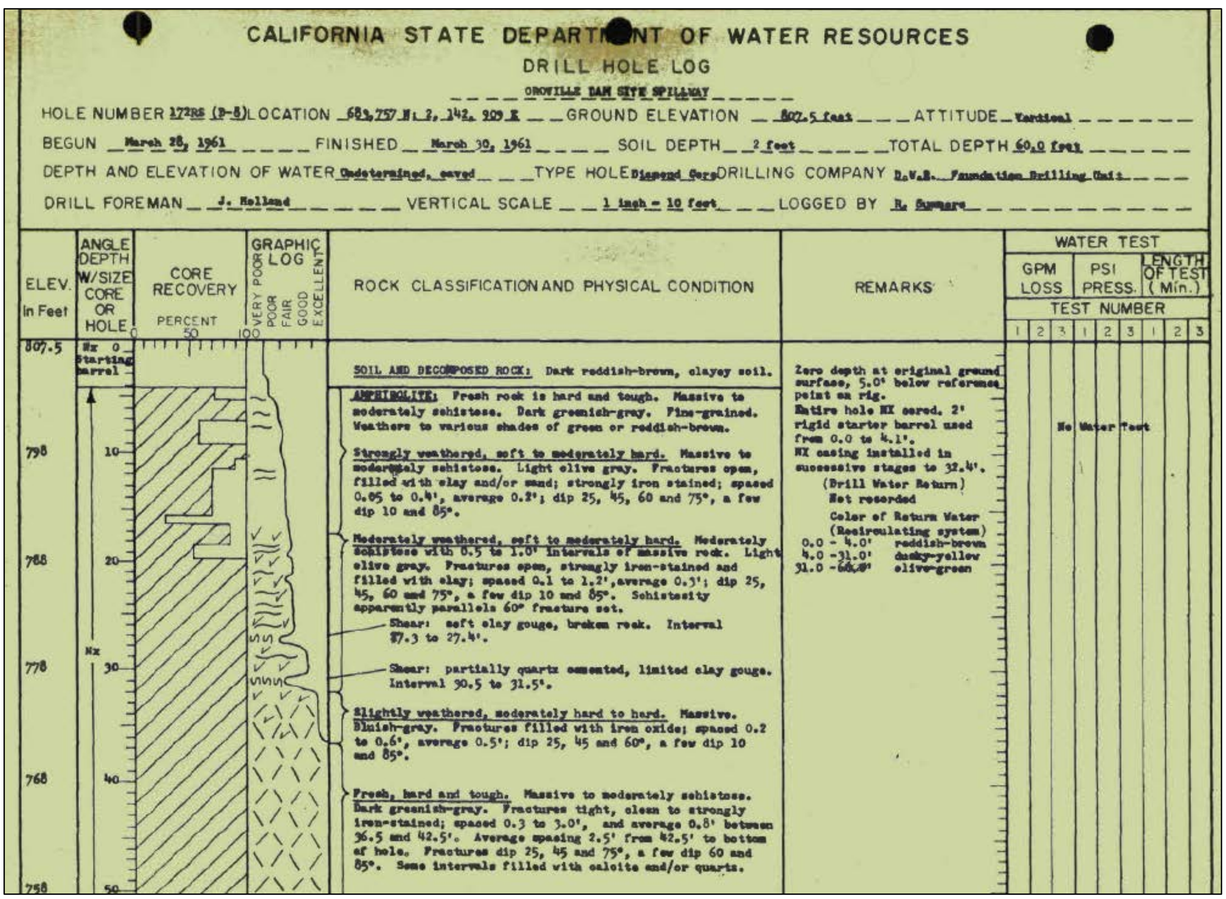
Detailed geological logs are given for all the diamond core holes. Logging at the time did not quantify overall rock quality, joints, and fractures, etc., as would be done today, however all descriptions are detailed and very precise. A general impression of rock quality was also given in a graphic chart, qualitatively ranging from very poor through excellent. Unfortunately, only an incomplete copy of this report was located, so that the legend sheet to describe this, and other various scales describing degree of weathering, jointing, etc. to accompany the logs was not available. However, descriptions on the logs are very detailed, and also included the depths to where the hole required steel casing (used to ensure the hole stayed open during drilling, and often a general indication of poor rock quality when required).

Even a cursory inspection of the borehole logs clearly indicates the significant depths of very poor to fair rock conditions. Table C-1, prepared by the IFT, summarizes the borehole logs, and indicates that the general depth of very poor to fair rock ranged from 7 to 40 feet, averaging about 22 feet. This is the same average depth of rock described on the logs as strongly weathered, soft, strongly jointed with open joints. The IFT believes that modern terminology would replace the term "strongly jointed" with the term "closely jointed." Regardless, in the experience of the IFT, any even relatively unexperienced geologist or geotechnical engineer today would immediately consider "strongly weathered, soft, strongly jointed, open joints" as conditions being very erodible, without the need for any special analysis or calculation. Table C-1 shows that these erodible conditions cover depths described as “Poor to Fair,” indicating that all such defined rock should be treated as erodible.

**Table C-1: Boreholes Along Emergency Spillway Alignment**

| Hole (xxxRS) | Approx. distance downstream from weir (ft) | Soil Depth (ft) | Strongly weathered, soft, strongly jointed, joints open | | Shear zones above slightly weathered rock | | Described as 'Poor to Fair ' | Comments |
|---|---|---|---|---|---|---|---|---|
| | | | to depth (ft) | to elevation (ft) | # | thickness (ft) | to depth (ft) | |
| 172 | 500 | 4 | 17 | 791 | 2 | 0.1, 1.0 | 17 | |
| 169 | 550 | 4 | 15 | 758 | 1 | 1.5 | 15 | |
| 170 | 800 | 3 | 10 | 746 | - | - | 17 | |
| 173 | 1100 | 1 | 10 | 707 | - | - | 10 | |
| 214 | 1400 | 5 | 13, 30 to 40, 52 to eoh at 65 | 667 | 1 | 1.2 | 40 | cased to 35 ft, fractures open where slightly to mod weathered at avg 0.15 to 0.4 ft spacing |
| 215 | 1700 | 4 | 9 | 619 | 1 | 0.05 | 9 | |
| 216 | 2100 | 7 | 0 | n/a | 1 | 0.8 | 7 | |
| 217 | 2400 | 11.5 | 16 | see comment | 1 | 20.0 | 33 | strongly fractured and sheared to 33 (elev 387) |
| 187 | 2600 | 4 | 37 | 374 | 3 | 4.1, 0.3, 1.3 | 37 | cased hole to 39.5 |
| 184 | 2900 | 4 | 30 | 357 | 8 | 0.8 to 3.0 | 30 | cased hole to 29.8 |
| 180 | 3200 | 5.5 | 22 | 345 | see comment | | 22 | major shear zone from 45 to 58.7 |
| 182 | 3650 | 5 | 10 | 284 | 5 | 1.0 to 2.4 | 10 | cased hole to 27 |

Figures C-6 and C-7 are taken from some of these borehole logs as examples.


CALIFORNIA STATE DEPARTMENT OF WATER RESOURCES
DRILL HOLE LOG
OROVILLE DAM SITE SPILLWAY
HOLE NUMBER 172RS (P-8) LOCATION 683,757 N; 2,142,909 E GROUND ELEVATION 807.5 feet ATTITUDE Vertical
BEGUN March 28, 1961 FINISHED March 30, 1961 SOIL DEPTH 2 feet TOTAL DEPTH 60.0 feet
DEPTH AND ELEVATION OF WATER Undetermined, caved TYPE HOLE Diamond Core DRILLING COMPANY D.W.R. Foundation Drilling Unit
DRILL FOREMAN J. Holland VERTICAL SCALE 1 inch = 10 feet LOGGED BY R. Summers

ELEV. In Feet | ANGLE DEPTH W/SIZE CORE OR HOLE | CORE RECOVERY PERCENT | GRAPHIC LOG (VERY POOR, POOR, FAIR, GOOD, EXCELLENT) | ROCK CLASSIFICATION AND PHYSICAL CONDITION | REMARKS | WATER TEST (GPM LOSS, PSI PRESS., LENGTH OF TEST (Min.), TEST NUMBER)

807.5, 798, 788, 778, 768, 758
Nx 0 Starting barrel; 10; 20; Nx 30; 40; 50

SOIL AND DECOMPOSED ROCK: Dark reddish-brown, clayey soil.

AMPHIBOLITE: Fresh rock is hard and tough. Massive to moderately schistose. Dark greenish-gray. Fine-grained. Weathers to various shades of green or reddish-brown.

Strongly weathered, soft to moderately hard. Massive to moderately schistose. Light olive gray. Fractures open, filled with clay and/or sand; strongly iron stained; spaced 0.05 to 0.4', average 0.2'; dip 25, 45, 60 and 75°, a few dip 10 and 85°.

Moderately weathered, soft to moderately hard. Moderately schistose with 0.5 to 1.0' intervals of massive rock. Light olive gray. Fractures open, strongly iron-stained and filled with clay; spaced 0.1 to 1.2', average 0.3'; dip 25, 45, 60 and 75°, a few dip 10 and 85°. Schistosity apparently parallels 60° fracture set.

Shear: soft clay gouge, broken rock. Interval 27.3 to 27.4'.

Shear: partially quartz cemented, limited clay gouge. Interval 30.5 to 31.5'.

Slightly weathered, moderately hard to hard. Massive. Bluish-gray. Fractures filled with iron oxide; spaced 0.2 to 0.6', average 0.5'; dip 25, 45 and 60°, a few dip 10 and 85°.

Fresh, hard and tough. Massive to moderately schistose. Dark greenish-gray. Fractures tight, clean to strongly iron-stained; spaced 0.3 to 3.0', and average 0.8' between 36.5 and 42.5'. Average spacing 2.5' from 42.5' to bottom of hole. Fractures dip 25, 45 and 75°, a few dip 60 and 85°. Some intervals filled with calcite and/or quartz.

Remarks:
Zero depth at original ground surface, 5.0' below reference point on rig.
Entire hole NX cored. 2' rigid starter barrel used from 0.0 to 4.1'.
NX casing installed in successive stages to 32.4'.
(Drill Water Return)
Not recorded
Color of Return Water (Recirculating system)
0.0 - 4.0' reddish-brown
4.0 - 31.0' dusky-yellow
31.0 - 60.0' olive-green

No Water Test

**Figure C-6: Borehole 172RS, approximately 500 feet downstream of Emergency Spillway Weir**

Although there is a general increase in rock quality with depth, there is great variability in the degree of weathering, jointing, rock strength etc., as would be expected. Rock quality (a product of all these different variables) is thus simply represented in a subjective graphic log.

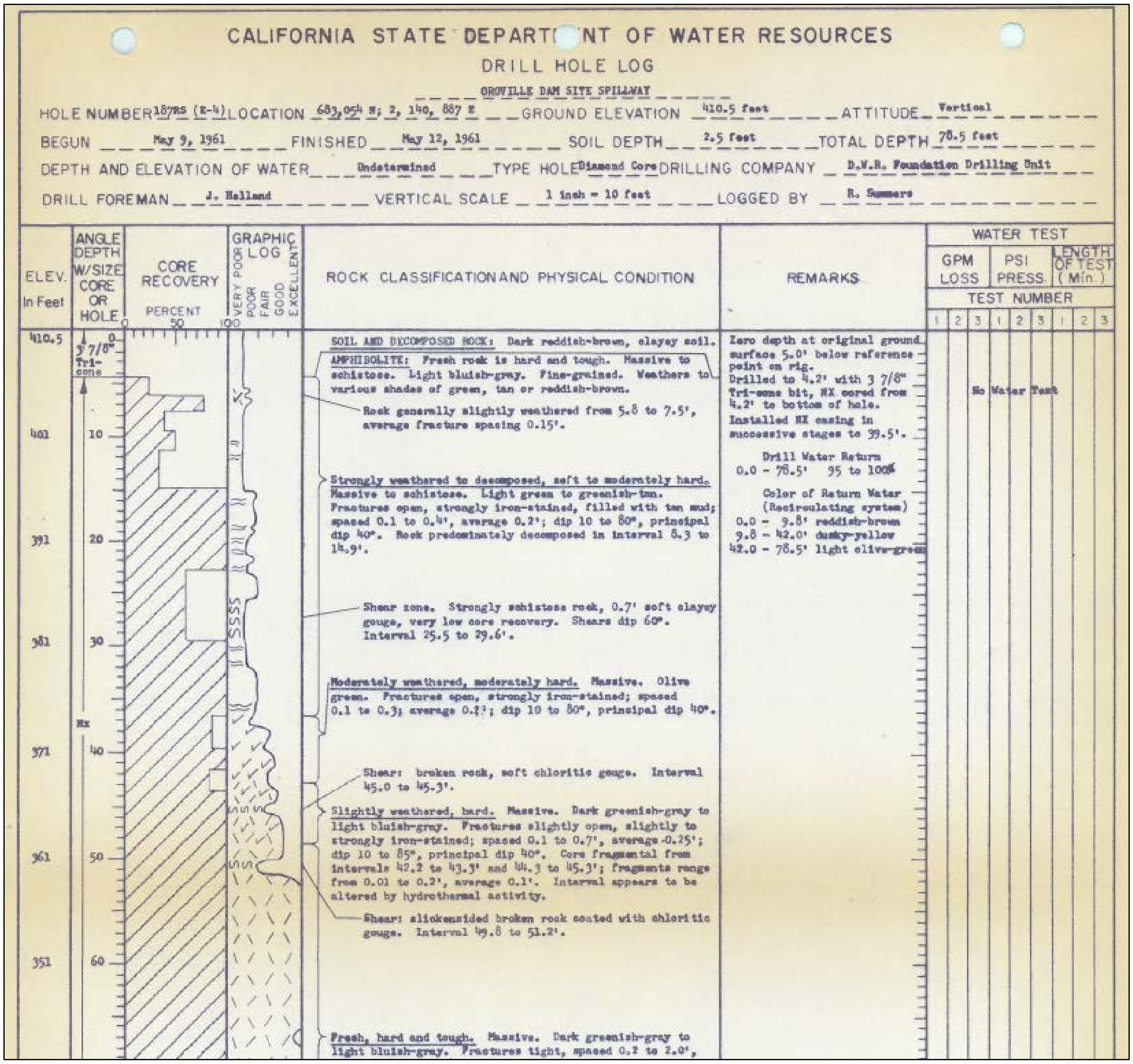



**Figure C-7: Borehole 187RS, approximately 2600 feet downstream of Emergency Spillway Weir**

In view of the high level of care and detail shown in the production of these borehole logs, it is difficult to believe that these authors did not consider these to be the conditions that they described as "subject to relatively accelerated erosion; where this is critical, the rock should be protected." As the invert is generally shown to be lower, on or into the "sound rock," there would be little or no rock requiring protection by a concrete lining. This could be the reason why the 1962 report shows the spillway excavation down into sound rock, and does not expressly address a concrete liner for the spillway channel. However, whether or not the invert elevation was originally designed as shown on this profile is unknown.

The IFT did infer through a memorandum [October 22, 1962, C-16] that the original concept design for the spillway had only a short length of lining and a large extent of unlined downstream channel. The memorandum expressed concern regarding this approach due to potential erosion, and expressed the belief that

> "… it should be mandatory that the spillway present no threat of artificially raising tailwater above normal."

Thus, downstream erosion was under consideration, but the possibility of headcutting back upstream to threaten the water-retaining crest structure itself was not. The concern regarding an unlined downstream channel was apparently heeded, as discussed in the next section.

### 3.4 Explorations for Service Spillway

In 1964, DWR completed additional investigations along the final alignment chosen for the service spillway. The 1962 report format was repeated in a new report entitled "Interim Exploration Data, Oroville Dam Spillway," [C-17] produced by many of the same personnel involved in the 1962 work.

The same degree of care and professionalism is evident in the production of the borehole logs, and geological descriptions are very similar. The IFT was given a complete copy of this report, including the legend sheet for the borehole logs that was missing from the 1962 report. A general impression of "foundation quality of rock for a concrete gravity structure" was given in a graphic chart, qualitatively ranging from very poor through excellent. Criteria considered in the classification of quality were given as:

> "Criteria for classification of the rock are rock weathering, hardness, schistosity, fracturing, and shearing. Several of these features are generally interrelated and are somewhat characteristic of a classification, although almost any one feature can have a dominant influence on the classification of the rock …"
>
> "Very Poor – Rock is characteristically very soft and usually friable. Generally, this class rock is found in sheared zones or in strongly weathered intervals.
>
> Poor – Rock is soft, but cohesive. This class is usually found in strongly to moderately weathered zones or some shear zones.
>
> Fair – Rock is moderately hard, generally coring in pieces 0.05 to 0.2 ft. in length. This class of rock is typically found in strong schistose zones or in slightly to moderately weathered zones.
>
> Good – Rock is massive to moderately schistose, and is fresh and hard except immediately adjacent to slightly weathered fracture planes. Rock generally cores in pieces 0.2 to 0.5 ft. in length.
>
> Excellent – Rock is fresh, hard, and relatively unfoliated; generally coring in pieces over 1.0 ft. in length."

It is evident that there was a full understanding that weathering is only one of a number of factors that determine the overall quality of the rock mass, and hence its performance. It is likely that this understanding was in place during the production of the earlier borehole logs, as the detailed geological descriptions in the borehole logs are quite similar to those from 1962.

A summary of the borehole logs based on the IFT's review is given in Table C-2. This is the entirety of the borehole information that was available during the development of the 1964 report. Along this new alignment, "depth of rock weathering varies from about 8 ft. to at least 52 ft. below the ground surface. Locally, weathering along shear zones may be even deeper."

Of the 10 borehole locations, two holes indicated good quality rock conditions at invert level, three holes terminated in good quality rock above invert elevation (which, as noted above, is not necessarily proof of continued sound rock with increasing depth), four holes (highlighted in yellow) indicated moderately weathered conditions, and one hole (highlighted in red) noted strongly to moderately weathered conditions (at about Sta. 22+00). Interestingly, the major shear at about Sta. 30+00 and downstream was not recognized at this time, due to the large spacing between boreholes.

It is noted that "Since drill hole data are more reliable than seismic data, 'Depth to Sound Rock' as presented in this report is based primarily on drill hole data." Sound rock is defined the same as in the 1962 reports, so that it is apparently based on its acceptability for use as riprap, although it was no longer the intent to establish a quarry in this area.

Based on the above information, a profile along the service spillway was developed, a portion of which is given below in Figure C-8. Note that, at the scale of the profiles, the elevations of both "sound rock" and the spillway chute invert as shown do not appear to correlate well with actual design conditions. It is possible that the geologists preparing these sections were not using finalized invert elevations. Also, as the profile was based primarily on drill hole data, in some cases more than 500 feet apart, the "sound rock" profile itself is quite unreliable, including in the area where the service spillway chute failure initiated. However the profiles do indicate what the geologists expected in terms of foundation conditions. In contrast to the 1962 profile along the older alignment, the spillway invert is now shown to be in both sound rock and overlying rock.

**Table C-2: Boreholes Along Service Spillway Alignment**
**See text for description of color highlights**

| Hole (xxxRS) | Approx Station (ft) | Offset from Centerline (ft) | Soil Depth (ft) | Strongly weathered, soft, strongly jointed, joints open | | Shear zones above slightly weathered rock | | Described as 'Poor to Fair ' | Approx Excavated Elevation (ft) taken from foundation cleanup maps (Final Geology Report) | Comments | Elevation at end of hole (ft) | Conditions at end of hole |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | to depth (ft) | to elevation (ft) | # | thickness (ft) | to depth (ft) | | | | |
| 290 | 16+00 | 140R | 2 | 23 | 840 | 1 | 1.8 | 27 | 794 | ? Likely fresh, hard, massive: Hole terminated above grade | 805 | 20 ft of fresh, hard, tight joints above end of hole |
| 291 | 17+00 | 130L | 0.5 | n/a | 858.5 | 1 | 0.02 | 7 | 787 | ? Likely fresh, hard, massive: Hole terminated above grade | 824 | 20 ft of fresh, hard, tight joints above end of hole |
| 289 | 17+00 | 140R | 2 | 27 | 816 | 1 | 2.0 | 27 | 787 | ? Hole terminated above grade | 799 | moderately to slightly weathered, tight to open, 0.05 to 0.8: |
| 286 | 18+50 | 94R | 2 | 38 | 781 | 2 | 1.0, 1.6 | 44 | 780 | moderately to slightly weathered, tight to open fractures, 0.05 to 0.4 spacing, just at contact with overlying strongly weathered zone | 766 | 4 ft of slightly weathered to fresh |
| 294 | 22+00 | 75R | 4 | 38 | 742 | 1 | 6.8 | 45 | 757 | strongly to moderately weathered, soft, strongly fractured another 15 ft to reach slightly weathered, fair to good quality, tight fractures | 713 | 12 ft of fresh, hard, massive |
| 283 | 26+15 | C | 3 | 23 to 45 | 725 | 3 | 1.0, 2.0, 2.2 | 60 | 722 | mod weathered, moderately hard, tight fractures 0.05 to 0.4 , major shear 5 ft below | 710 | 4 ft of fresh, hard |
| 282 | 31+00 | C | 2 | 23 | 633 | 2 | thin | 23 | 615 | fresh, hard, massive | 616 | 10 ft of fresh, hard |
| 278 | 33+00 | 103R | 3 | 36 | 572 | 4 | 0.5 and less | 36 | 567 | at very bottom of strong weathered zone: moderately weathered, moderately hard, fair, tight to open fractures | 542 | 10 ft of fresh, hard |
| 277 | 38+25 | 80L | 6 | n/a | n/a | 2 | 0.5, 0.3 | 6 | 437 | slightly to moderately weathered, tight to open, 0.05 to 0.8 | 406 | 11 ft fresh hard massive |
| 275 | 42+70 | C | 4 | 31 to 43 | 342 | 2 | 1.5, 1.5 | 45 | 327 | fresh, hard, massive | 322 | fresh, hard, massive |

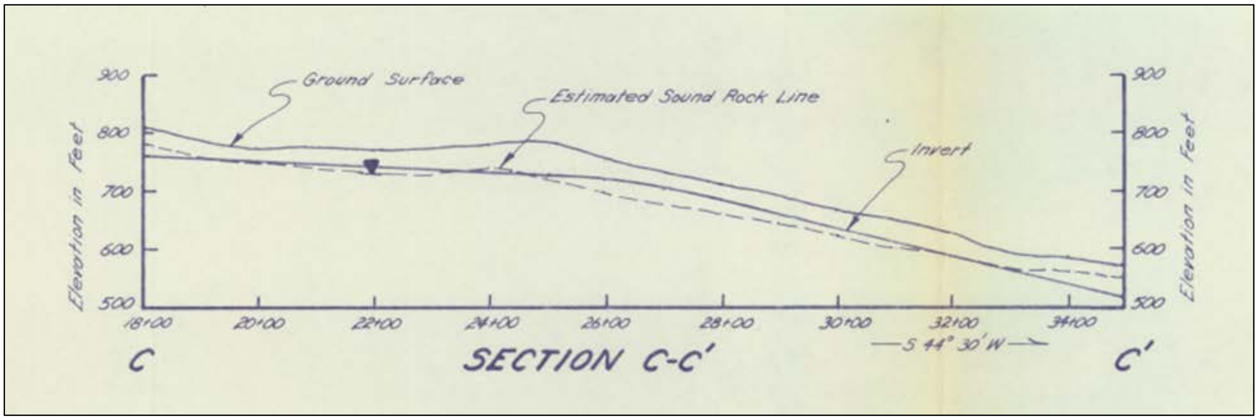



**Figure C-8: Portion of a longitudinal section along final service spillway alignment, based on 1964 investigations.**

There appears to have been a major change in opinion regarding the geotechnical acceptability of moderately weathered rock. The 1964 report states that:

> "Moderately weathered rock should also be adequate foundation for most of the structures, but may require some special treatment. Irregularity of foundations excavated in fresh or slightly weathered rock will provide good keying against sliding of the concrete apron or lining. Fresh or slightly weathered amphibolite is also very resistant to scouring and/or plucking."

The IFT believes that this major change in opinion may be due to the change in design that now included a concrete liner over the entire length of the spillway channel. The liner appears to have been added to the design in response to the concerns raised in October 1962, and thus is apparently to protect the moderately weathered rock.

The above statements cannot be reconciled with the fact that the exploratory boreholes showed worse than moderately weathered rock conditions at the spillway invert elevation. Figure C-9 clearly shows that strongly weathered rock conditions would have been expected at the level of the spillway chute excavation. However, it may be that the invert elevation was modified from that under consideration at the time of the development of the profile given in Figure C-9.

No mention is made of potential rock scour of weathered rock, as was documented in 1962; only the original observation regarding fresh and slightly weathered rock is repeated in 1964. In this later report, it is noted that moderately weathered rock "may require some special treatment," but this special treatment is not defined further.

**Excavation and Drainage Considerations**: As the profile clearly shows moderately weathered rock as an acceptable foundation condition, the text opines on how it will be excavated – being either blasted with explosives or ripped out by heavy machinery. However, a basic inconsistency arises in the text in regard to the rippability versus blasting of the moderately weathered rock, an inconsistency that became the basis of a major claim during construction. Most moderately weathered rock is described as rippable:

> "Rippable material will include all overburden, all decomposed and strongly weathered rock *and most moderately weathered rock.* Average depth of rippable rock is about 20 ft …" (italics added for emphasis)

Since the profile shows a large portion of the chute founded on moderately weathered rock, it could be inferred that this portion of the chute would be rippable. However, it is inferred (but not explicitly stated) that where the spillway chute cannot be founded on sound rock, it should be founded on *non*-rippable rock:

> "The lining (walls and invert slab) of the spillway chute should be founded on sound rock wherever possible, but most of the non-rippable rock will be adequate for this structure"

In the following excerpt, there is a possible explanation for this apparent contradiction:

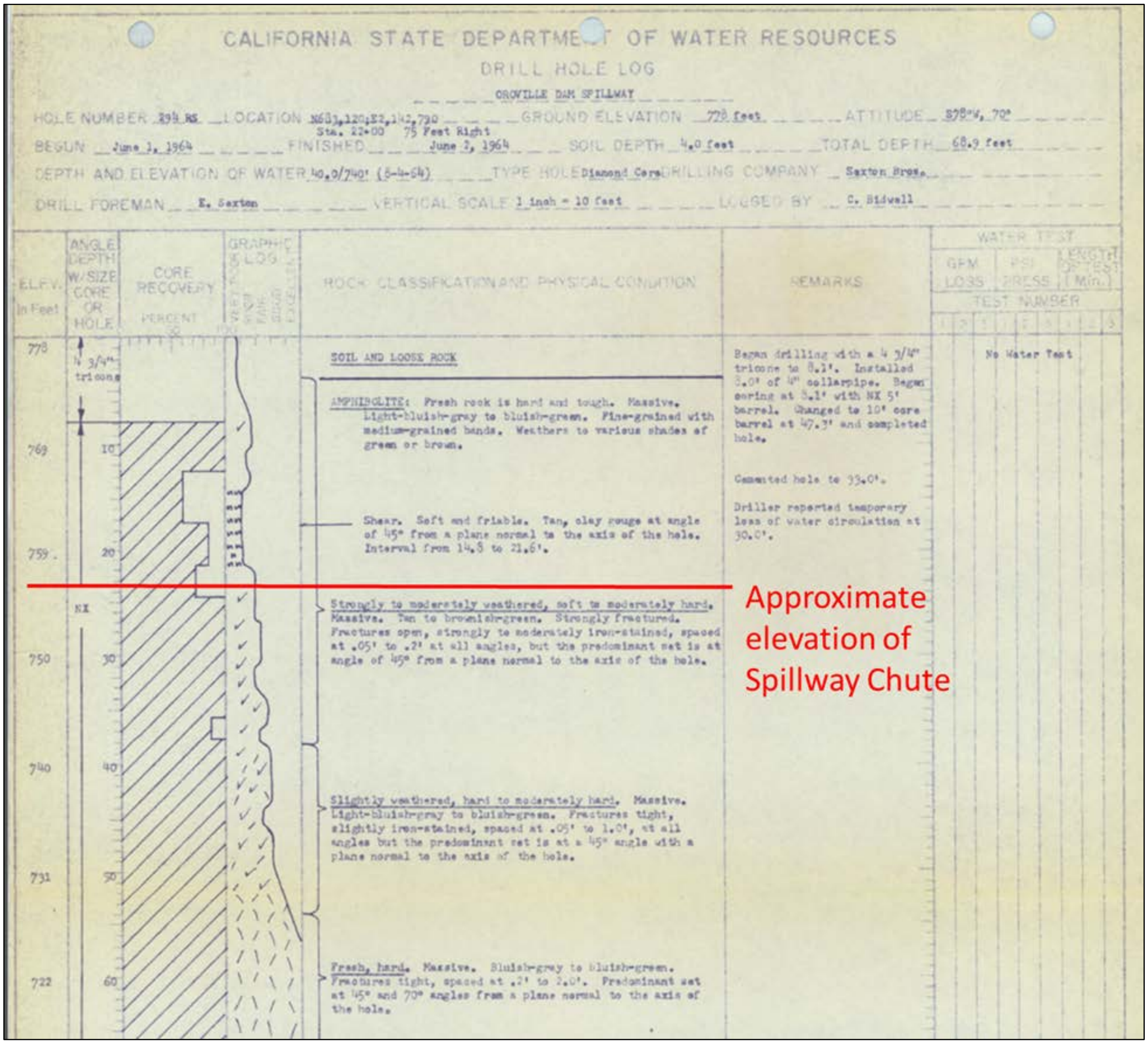
> "Sound rock (fresh and slightly weathered rock that is relatively massive and slightly to moderately fractured) will require drilling and blasting for removal. Blasting will also be required to excavate portions of the moderately weathered rock. Because protruding ribs of hard rock (sic), it may be advantageous to blast much of the rippable, moderately weathered rock to facilitate excavation."

It appears that the authors considered that *blasting* of rippable moderately weathered rock would produce an acceptable foundation that would not require special treatment, perhaps due to the heterogeneous mix of weathered and non-weathered rock.

It is evident that only the strongly to moderately weathered material encountered in Borehole 294RS (Sta. 22+00) was considered as being rippable:

> "Rippable material at and below invert grade should be anticipated in the chute between Stations 19+00 and 23+00."

The detailed borehole log for 294RS, located closest to this area, is given in Figure C-9.



CALIFORNIA STATE DEPARTMENT OF WATER RESOURCES
DRILL HOLE LOG
OROVILLE DAM SPILLWAY

HOLE NUMBER 294 RS LOCATION N683,120;E2,142,790 Sta. 22+00 75 Feet Right GROUND ELEVATION 778 feet ATTITUDE S78°W, 70°
BEGUN June 1, 1964 FINISHED June 2, 1964 SOIL DEPTH 4.0 feet TOTAL DEPTH 68.9 feet
DEPTH AND ELEVATION OF WATER 40.0/740' (6-4-64) TYPE HOLE Diamond Core DRILLING COMPANY Sexton Bros.
DRILL FOREMAN E. Sexton VERTICAL SCALE 1 inch = 10 feet LOGGED BY C. Bidwell

| ELEV. In Feet | ANGLE DEPTH W/SIZE CORE OR HOLE | CORE RECOVERY PERCENT | GRAPHIC LOG | ROCK CLASSIFICATION AND PHYSICAL CONDITION | REMARKS | WATER TEST: GPM LOSS / PSI PRESS / LENGTH OF TEST (Min.) |
|---|---|---|---|---|---|---|
| 778 | 4 3/4" tricone | | | SOIL AND LOOSE ROCK | Began drilling with a 4 3/4" tricone to 8.1'. Installed 8.0' of 4" collarpipe. Began coring at 8.1' with NX 5' barrel. Changed to 10' core barrel at 47.3' and completed hole. | No Water Test |
| | | | | AMPHIBOLITE: Fresh rock is hard and tough. Massive. Light-bluish-gray to bluish-green. Fine-grained with medium-grained bands. Weathers to various shades of green or brown. | | |
| 769 | 10 | | | | Cemented hole to 33.0'. | |
| | | | | Shear. Soft and friable. Tan, clay gouge at angle of 45° from a plane normal to the axis of the hole. Interval from 14.8 to 21.6'. | Driller reported temporary loss of water circulation at 30.0'. | |
| 759 | 20 | | | | | |
| | NX | | | Strongly to moderately weathered, soft to moderately hard. Massive. Tan to brownish-green. Strongly fractured. Fractures open, strongly to moderately iron-stained, spaced at .05' to .2' at all angles, but the predominant set is at angle of 45° from a plane normal to the axis of the hole. | | |
| 750 | 30 | | | | | |
| 740 | 40 | | | | | |
| | | | | Slightly weathered, hard to moderately hard. Massive. Light-bluish-gray to bluish-green. Fractures tight, slightly iron-stained, spaced at .05' to 1.0', at all angles but the predominant set is at a 45° angle with a plane normal to the axis of the hole. | | |
| 731 | 50 | | | | | |
| 722 | 60 | | | Fresh, hard. Massive. Bluish-grey to bluish-green. Fractures tight, spaced at .2' to 2.0'. Predominant set at 45° and 70° angles from a plane normal to the axis of the hole. | | |



**Figure C-9: Borehole 294RS, approximately Sta. 22+00 along Service Spillway, offset 75 feet**

Although rippable conditions were called out in the report, apparently associated with strongly to moderately weathered rock of poor to fair quality (as per Figure C9), apparently no special treatment for such fully rippable foundations was considered, and the following general statement was given:

> "Excavation of the chute will produce a rough, irregular rock surface and shear keys are not thought to be necessary to prevent sliding of the chute lining. Blasting for shear key trenches in the foundation would only tend to weaken the rock by shattering."

The IFT does believe that this statement is valid. However shear keys could have been constructed by other less destructive methods, such as the use of jackhammers, as discussed further in Appendix E. This statement may have been the basis for not providing foundation keys even in areas where the foundation was in strongly and moderately weathered rock.

The report also notes the potential for uplift pressures; the main concern being leakage through the foundation past the grout curtain at the upstream end of the spillway chute. It recommends an underdrain system along the entire chute:

> "The possibility of uplift pressure is most likely to occur near the headgate, but could occur at any point on the chute. A close-spaced drain-hole system should be provided in the vicinity of the headgate structure, immediately downstream from the grout curtain, to reduce the possibility of uplift pressures on the chute invert slab. A drainage ditch paralleling each side of the chute, with longitudinal and lateral drains in or under the slab should prevent damaging pressures from developing. The longitudinal underdrains should be spaced no more than 30 ft. apart, and the lateral underdrains should be spaced no more than 60 ft. apart. Underdrains should be designed which would eliminate the necessity of blasting trenches in the rock. Installation of rock anchor bars would provide added protection against damage due to uplift pressure and subsequent destruction of the slab by high velocity flow."

The report fails to connect the geological descriptions of the "rippable" moderately weathered rock with the potential for scour of this "rippable" rock due to flow in the underdrains. One wonders whether, if the 1962 comments on potential scour had been carried forward to the 1964 report, this now-obvious connection would have been made. Regardless, it is evident that there was no intention of placing the underdrains on a *strongly* weathered rock foundation, and that blast damage to the foundation rock was to be avoided, both likely in recognition of potential damage due to seepage flows.

**Geophysical Information**: The 1964 report also includes a summary of both the 1961 and 1963 geophysical investigations. A summary plot of the earlier investigations along the alignment of the emergency spillway is given in Figure C-10.

This summary shows large depths to sound rock, as defined for the purpose of riprap exploitation. The relevance to the current discussion is that the report indicates that there was no discernable difference in velocity from the surficial soil and decomposed rock layer until the "sound" rock layer was reached in most surveys, i.e. the measured velocities of the moderately to strongly weathered rock were not different enough from the measured velocities of the soil and decomposed rock to be interpreted as a separate layer. This would be yet another indication that significant depths of erodible material could be expected downstream from the emergency weir.

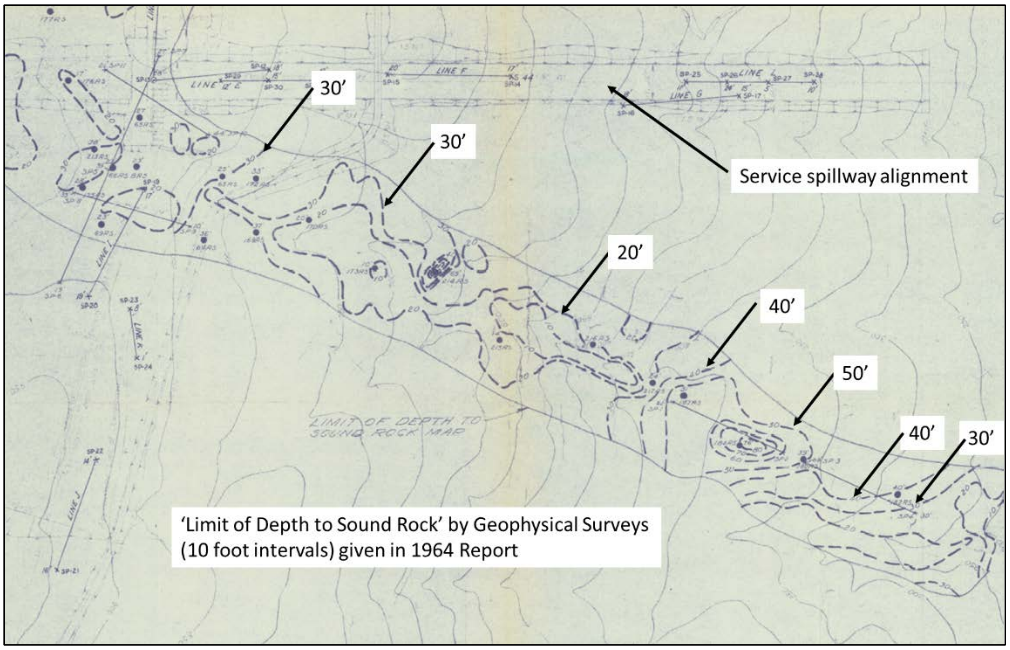



**Figure C-10: Summary of 1961 Geophysical Results downstream from current emergency spillway weir**

## 3.5 Specifications

Based on the above reports, the design and specifications for the work were developed. The contract specifications[C-18] for spillway excavation are quite brief; relevant sections follow:

> "Surfaces against which concrete is to be placed shall be cleaned of all loose and objectionable material by means of high velocity air or air and water jets, or by other approved means …"

> "Shear zones extending below the excavation payment lines for concrete structures shall be excavated to remove decomposed, weathered, or crushed rock and similar unsuitable foundation materials, as directed. Excavation shall include the use of pneumatic tools, hand barring and wedging …"

> "Excavation for the chute shall be to fresh or moderately weathered rock that *cannot be further removed by heavy duty power excavating equipment*, but shall be excavated to a minimum of the excavation limit lines shown on the drawings, regardless of the excavation methods used. Sharp points of undisturbed rock will be permitted to extend not more than 3 inches within the excavation limit lines provided they do not occur along the alignment of foundation drainage pipes …" (italics added for emphasis)

Note that the specifications do not cover the conditions in which moderately or strongly weathered rock *can* be easily removed without heavy duty power excavating equipment beyond the excavation limit lines.

### 3.6 1965 Design Review

On January 12, 1965, an internal DWR civil/structural design review of the spillway was documented by J. E. Halstead [C-19]. Foundation conditions were apparently well understood:

> "The foundation consists of amphibolite rock. This rock is hard when fresh or slightly weathered and very soft when crumbly or decomposed. The soil zone usually grades down into decomposed rock and the rock is generally progressively less weathered with depth. However, there are places where the weathering extends down into and beneath the rock outcroppings along shear and strongly fractured zones. Deep weathering pockets may exist between rock outcrop areas.
>
> The specifications provide for excavating to rock which is no less competent than moderately hard or moderately weathered. Therefore, backfill concrete will probably be required for the purpose of filling deeply weathered pockets. The flood control structure is founded on sound rock."

However, it does not seem to be recognized that the specifications do not adequately cover the situation where "backfill concrete will probably be required for the purpose of filling deeply weathered pockets."

### 3.7 Conditions Encountered During Construction

The omission noted above, related to deeply weathered pockets, led to significant issues arising during final inspections of the spillway chute foundation prior to placement of the concrete; from the 1968 Spillway Final Construction Report [C-20]:

> "Excavation for the chute shall be to fresh or moderately weathered rock that cannot be further removed by heavy duty power excavating equipment." The Contractor considered that this statement gave him the right to overexcavate material several feet below grade and then get paid for backfill concrete at $30.00 per cubic yard to restore the subgrade to the elevation shown on the drawings. The Field Engineer directed the Contractor to excavate to the grades shown on the drawings unless ordered otherwise by himself or one of his subordinates …"

It is not stated in this report whether the moderately weathered rock could be easily removed to below invert elevation, although this was most certainly the case according to daily reports during construction. The following statement by DWR appears in documentation associated with Change Order #21 [C-21], relating to a major claim by the Contractor due to the bedrock conditions encountered in the chute foundation:

> "Originally our position has been that payment and any excavation with heavy power equipment beyond the minimum excavation limit lines would be considered as overexcavation. However, after discussion with the Contractor and our field personnel, it was determined that it was reasonable for the Contractor to take literally Subarticle (5) and to expect to excavate at least all shear zone material that could be removed by heavy duty power excavating equipment. Accordingly, it was agreed that payment was due for all the

> shear-zone excavation and the concrete backfill costs for that material so excavated by heavy duty power equipment."

Note that the above pertains specifically to shear zones, which were carefully mapped and numbered, and to material excavated by heavy duty power equipment. It does *not* apparently include areas of moderately or highly weathered rock unless excavated by heavy duty power equipment. However, removal of *all* the highly weathered rock was certainly the original intent of the field geologists, and would have been required to meet the specification noted above whereby surfaces were to be cleaned of "all loose and objectionable material."

In notes from a meeting held on September 6, 1967 [C-22] in which the claim was settled, the following is stated:

> "Mr. Mims (Contractor representative) then stated, I spent $1,200,000 over the payments made to me for the work on the chute… The foundation rock was shattered and could be excavated with a monitor to a depth of 25 ft. I have an independent geology report and colored photos of every inch of the bottom. Dynamiting breaks the rock lower by reason of numerous fractures. We stopped at grade but couldn't control raveling out below grade… At this point Mr. McCune (DWR) entered the discussion stating that you drilled and shot the chute, but the overbreak was due to your methods and nothing else… Mr. Mims countered with the statement that you have to assume the geological information was correct… The geological information did not resemble what was out there."

It is clear that portraying foundation conditions at the invert as being acceptable was central to DWR's defense against the Contractor's claim. It is possible that the field geologist was over-ruled by the project office during the foundation preparation in the area of 33+00, where there was no means to deal with the as-found conditions within the contract, and moderately to strongly weathered rock was left in place. However, this remains speculation.

It also remains speculation as to whether a DWR corporate reluctance to acknowledge out-of-specification foundation conditions became entrenched as part of oral organizational history. Alternatively, institutional memory may have been overly influenced by assertions in the report that implied more favorable foundation conditions than expected and also by the title of the report which includes the word "Final" under the assumptions that all previous observations and interpretations had been taken into account. From the Spillway Final Construction Report[C-19]:

> "In the chute, there was very little extra excavation directed. This consisted of a few clay seams in the foundation and the areas where the slope failures occurred."

Daily reports of construction activities were examined by the IFT, and these reports support the Contractor's claim of poor foundation conditions, and not necessarily the comment regarding direction by DWR for extra excavation. This is discussed further in Appendix A.

In regard to the rock anchors in the chute, the following excerpts are from the Spillway Final Construction Report[C-19]:

> "In order to determine what depths would be necessary to obtain the required tension (of the chute anchors), three sets of two holes were each drilled in the chute foundation; two

> 5-foot holes, two 6-foot holes, and two 7-foot holes. In each case, one hole was located in the worst foundation available, that is, clay seams, and the other hole was for average conditions. It was determined that the minimum depth of 5 feet would produce over 30,000 psi regardless of the type of foundation."
>
> "… Other problems were loss of holes due to the fractured rock and keeping the holes free of water."

The report either did not recognize, or chose not to comment, on the fact that the anchor tests were not conducted in the worst foundation, but rather the "worst foundation available" at the time of the tests (they included sections with clay seams), which were completed near the start of construction, and would not have included large areas of strongly weathered rock. Later problems noted due to loss of holes in fractured rock would indicate worse conditions than were tested, however, the assertion that the 5-foot anchors were sufficient "regardless of the type of foundation" remained unchallenged.

### 3.8 Board of Consultants Involvement

DWR convened an external Board of Consultants (BOC) during the design and construction of Oroville Dam. This BOC consisted of six members reporting directly to the Chief Engineer of DWR, and covered the design and construction aspects of all the Oroville facilities.

In review of the available BOC documents, there are very few comments covering the spillway chutes. However, comments from an April 1963 meeting [C-23] appear to support the IFT's belief that the concrete lining was only added to the design after the 1962 Interim Report on Geologic Investigation.

> "… (the BOC) concurs generally in the selection from among other alternatives of a concrete lined chute …"

One interesting comment appears regarding the area downstream of the emergency spillway. Notes from a September 20, 1963 meeting [C-24] include the passage:

> "The Board concurs in the finding that nothing be done to minimize erosion in the natural channel downstream from the auxiliary spillway beyond ensuring that this discharge be kept away from the concrete lined chute below the flood control outlet structure."

Of note regarding the service spillway, a memorandum [C-25] notes that the "foundation treatment in the weathered rock area around Station 31+00" was discussed in an April 1966 BOC meeting. After an exhaustive search by DWR staff, unfortunately the minutes of this meeting could not be located. However, it is apparent that during the ensuing discussions, the adequacy of the underdrain system was questioned, and this led to a Change Order (#21), issued October 16, 1967, as previously discussed. All of the drain pipe sizes were increased, the herringbone pattern was developed to provide positive drainage throughout, and vertical risers were added to the longitudinal drains.

## 3.9 Foundation Conditions as Documented in Final Geologic Report

A Final Geologic Report [C-1] was prepared in 1970 for the Spillway, following completion of construction. The report is under the signature of J. W. Marlette, a new Chief Project Geologist.

> "Periodic inspections of foundation, excavation and grouting were made by D.J. Gross, who made recommendations as required. Daily inspections of rock conditions encountered during excavation were made by O. L. Huber."

Both Gross and Huber were involved with the 1964 investigations; but apparently Gross was the only geologist that was involved throughout the project from the 1962 investigations.

The report covers the service spillway only; the only mention of the emergency spillway is in the Introduction:

> "The ungated weir (emergency spillway), which will probably never be used, provides assurance that Oroville Dam will ever be overtopped."

It is clear that documentation of geologic conditions downstream of the emergency overflow weir was considered to be relatively unimportant, and that use of the emergency spillway for cases other than extreme floods was not being considered.

The report is very detailed and provides an excellent record of as-found conditions along the service spillway chute, including:

- Foundation geological mapping sheets showing Geologic Units and Engineering Properties, including 124 specific shear zone tabulations along the chute, and
- Detailed drawings showing cleanup of the chute foundation, where the foundation rock was left partly covered with " compacted clayey fines."

Full descriptions of the as-found weathered rock are included, and the factors that affected the quality and appearance of the foundation rock are also well explained:

> "Even though there is only one rock type exposed in the spillway foundation, several factors affect the quality and appearance of this foundation rock. Foremost among these factors is the degree of weathering of the rock at final grade. Other factors which affect the quality of the foundation to a lesser degree are (1) spacing and orientation of joints and shears, (2) thickness, orientation and composition of materials in the wider shear and schist zones (3) the presence of strongly fractured or crushed zones (many of which were created by blasting, as shown in Photo 123), and (4) the degree of schistosity or foliation exhibited by the rock."

The mapping details the areas where strongly weathered rock was encountered, described as

> "Strongly weathered to decomposed rock – soft, samples easily broken by hand, dull thud when struck with hammer, most of the minerals are partially or entirely altered"

There is no moderately or strongly weathered rock noted in the foundation for the first 600 feet downstream from the headworks structure. The most upstream locations where such conditions were found are documented on the right side of the chute near Sta. 19+00 to 20+50, followed by an area on the left side at Sta. 27+00, with the largest areal extents from Sta. 31+00 to 38+00, in the area of the initial failure in February 2017. Figures C-11 and C-12 are excerpts from drawings in the Final Geologic Report showing the large extents of the moderately and strongly weathered rock, and the degree of compacted clayey fines left upon foundation cleanup in the vicinity of the initial spillway slab failure (colorized by the IFT). These aspects are discussed further in Appendix I.

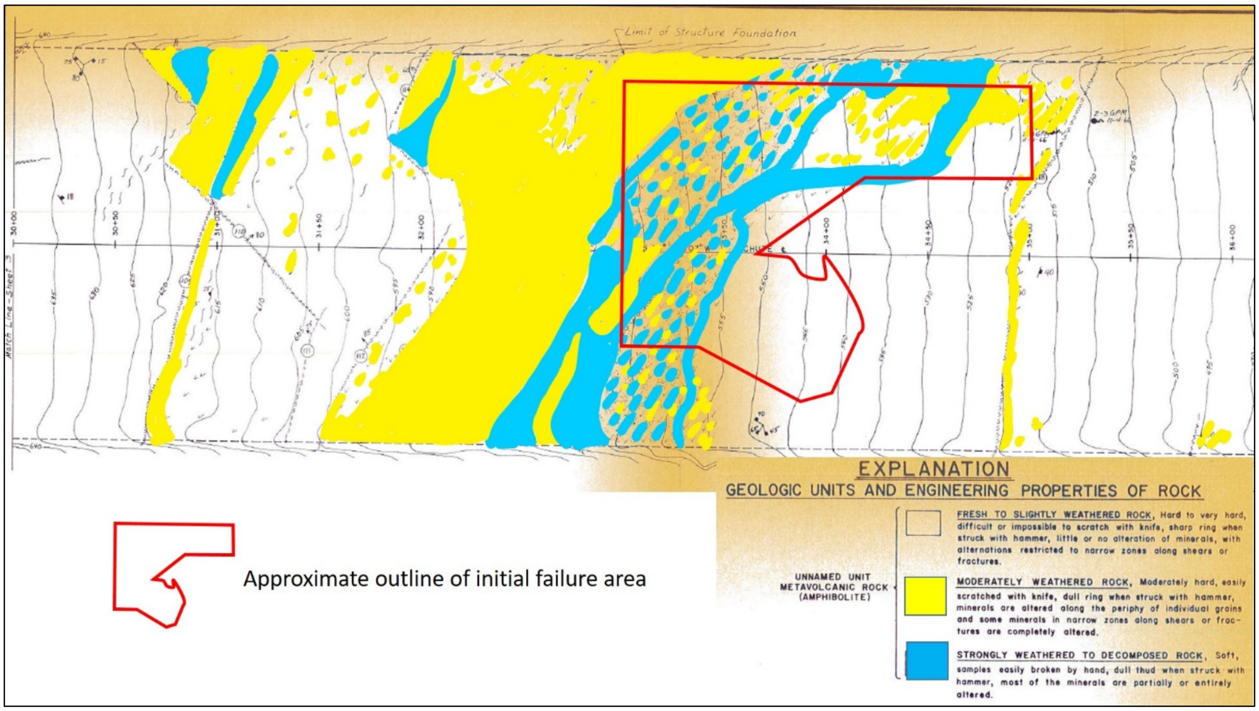



**Figure C-11: Extents of Moderately and Strongly Weathered Rock. Geologic mapping from Final Geologic Report [C-1]**

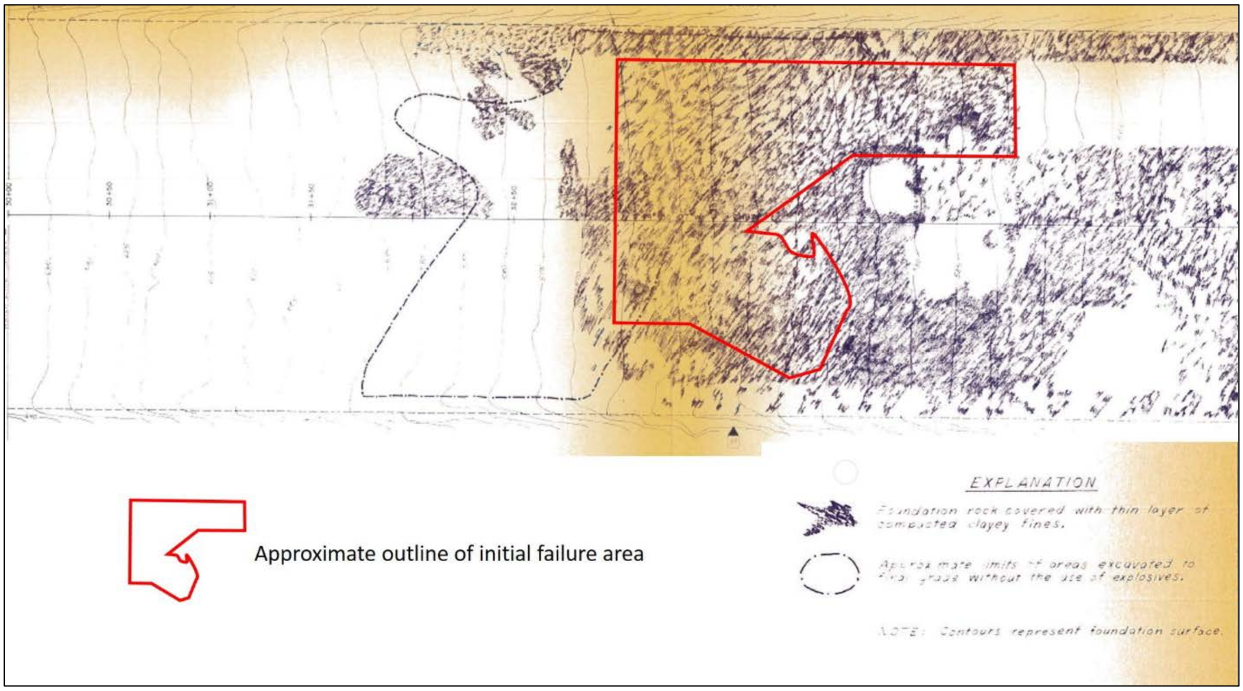



**Figure C-12: Extents of Compacted Clayey Fines. Geologic mapping from Final Geologic Reportm [C-1]**

Encountering strongly weathered rock was clearly never expected at the time of the 1964 Interim Geological Report. Although there are references in this report back to the early reports, there is no apparent connection made to the fact that these obvious changed foundation conditions were not covered under the project specifications.

Groundwater is noted as being encountered:

> "During excavation of the spillway chute, a few such springs were encountered (Plate 2), but none created a problem during construction, because the flows were very small (2 to 6 gpm). The chute slab underdrains should prove adequate to handle any future flow from these springs."

It is clear that the underdrain system was seen as necessary in response to potential groundwater and seepage past the grout curtain, and water injection during spilling was not being considered. Gravel-filled sonotube drain risers are shown in photographs (see Appendix A) as being installed in areas of overexcavation, but not commented upon in the text. Again, it is clear that the drains were mainly, or solely, included in the design to deal with potential seepage from the bedrock.

The following passage notes that the main consideration in foundation cleanup was bearing strength and groundwater seepage requirements, not erodibility:

> "The foundation underdrains beneath the chute lining slab consist of a network of six-inch perforated V.C.P. tile drains on a herringbone pattern, covered with select gravel backfill. In some areas these drains were placed on rock which had been thoroughly cleaned up, but in other areas only the loose material was removed from the foundation rock, leaving a

> layer of compacted clayey material on the rock beneath the drains (Photos 37 through 39, Plate 3). Removal of the compacted clayey material would not be necessary to satisfy foundation strength requirements for the chute lining. Also, removal of this material from 100 percent of the foundation would not be necessary to satisfy underdrain requirements. Therefore, the goal set by design engineers (removal of fines from 50 percent of the foundation) was a practical one, which would provide adequate relief of seepage pressures."

The IFT could find no other reference to the decision (by the design engineers or others) to relax the specifications and allow 50% fines to remain in the foundation, nor was it ascertained over what unit base area this estimation would be made.

A general statement near the beginning of the report states:

> "Foundation rock for the entire spillway is amphibolite, which contains numerous narrow shears and schistose zones. Fresh amphibolite is hard, dense, fine- to medium-grained, greenish gray to black, and generally massive, although a slight foliation (regional structure) is usually present."
>
> "Joint spacing, within each of the three major joint sets, ranges from a fraction of an inch to several feet, and averages about two feet."

This opening description, when read in isolation from the remainder of the report, describes very favorable foundation conditions, and this may be a major factor in the ensuing misinterpretations of bedrock conditions.

### 3.10 Foundation Conditions Documented After Construction

A major publication, "Bulletin 200," covering the execution of the entire project was published by DWR in 1974 [C-26]. However, there are very few passages pertaining to spillway chute conditions:

> "Rock at the site is moderately to strongly jointed and is transected by steeply dipping shears and schistose zones … The depth of weathering was found to be substantial and varied greatly from place to place …
>
> Except for a narrow strip immediately downstream from the weir, the terrain below the weir was not cleared of trees and other natural growth because emergency spillway use will be infrequent …
>
> Approximately 90% of the chute foundation required blasting to reach grade. The only extra excavation directed was removal of a few clay seams in the foundation and a few areas where slope failures occurred."

There is no mention of the changed conditions that were encountered during the service spillway chute excavation and the ensuing contractual and technical issues.

Of particular interest is the note that stripping of the terrain below the emergency weir was thought not to be required because of its infrequent use. Bulletin 200 states that the service spillway maximum discharge was chosen to limit Feather River flows to downstream levee channel capacity during the project design flood. The probability recurrence interval of this flood was given as approximately 450 years. Thus, the designer expected the likelihood of emergency spillway use to be in the order of 1:450 in any one year, or about a 20% chance over 100 years of project operation. This may have had implications in the assessment of failure modes as discussed in Appendix F3.

### 3.11 2002 Comments on Use of Emergency Spillway

The IFT was provided with no records of any geologic reviews or documentation from Bulletin #200 (1974) up until 2002. In August 2002, the Yuba County Water Agency produced "Technical Memorandum on Controlled Surcharge of Lake Oroville for Additional Flood Control" [C-27]. This memo was apparently not entered into the administrative record, but a cut-and-paste version of the memo, taken from a Yuba County Water Agency web page available at the time, was provided to the IFT. From the provided version:

> "The objective of this technical memorandum is to document the findings of studies under the Water Act of 2000 regarding the emergency operation of Lake Oroville for additional flood control."

The memorandum was focused on a proposed plan involving drawdown of the Thermalito Afterbay for supplemental flood control, and the memorandum included considerations of the emergency spillway weir. The memo provides comments on the impacts of using the emergency spillway as follows:

> "The discharge area below the emergency spillway is not armored and extensive erosion would take place if the emergency spillway were used. The spillway road and possible high voltage transmission towers would be impacted."
>
> "Because the area downstream from the emergency spillway crest is an unlined hillside, significant erosion of the hillside would occur."
>
> "EMERGENCY SPILLWAY IMPACTS: The hillside between the emergency spillway and the Feather River would be subject to severe erosion when water flows over the spillway. Depending on the rate of flow, the erodable area, as generally indicated by contours on Figure 1, could range from 50 to 70 acres. The amount of soil, rock, and debris that would fall into the Feather River could be very large, depending on the depth of erosion. There could be damages to downstream structures, including the Thermalito Diversion Dam and Powerplant, Fish Barrier Dam, and highway bridges. If there is river channel blockage below the spillway, there could be impacts on operation of Hyatt Powerplant. Additionally, erosion of 50 to 70 acres down to bare rock would have a significant adverse visual impact and effects on birds and wildlife that occupy the area."

### 3.12 2005 Review of Geologic Conditions at Emergency Spillway

As part of the Oroville re-licensing process, a Motion to Intervene [C-28] was submitted to the FERC on October 17, 2005 by three environmental groups – Friends of the River, the Sierra Club and the South Yuba River Citizens League. One of the issues brought before the Commission in the Motion was directly based on the 2002 Yuba County memorandum, and cited the above quotations. On this basis, one of the requests to the FERC was:

> "Consistent with the Commission's responsibilities ... requiring relicensing applicants to demonstrate that existing structures are safe and adequate to fulfill their stated functions, [FERC should] issue a licensing order requiring the licensee to armor or otherwise reconstruct the ungated spillway and to make any other needed modifications so that the licensee can safely and confidently conduct required surcharge operations consistent with the Corps of Engineers Oroville Dam Reservoir Regulation Manual."

The November 27 edition of the Sacramento Bee newspaper carried a story covering claims made in the Motion to Intervene, including:

- The presence of a "design flaw" such that the emergency spillway "empties onto a bare dirt hillside"
- That "water flowing over the emergency spillway would wipe out two roads and two power lines built on the hillside below, and wash an estimated 70 acres of soil and rock downstream."

On November 29, the FERC requested an official response from DWR, which responded the same day (November 29) via a series of 17 emails [C-29]. It is clearly evident that the effort was rushed, with external pressure being applied to respond as quickly as possible. The IFT was not able to discern whether this pressure came directly from the FERC, or whether it came from DWR executives wishing to be seen as responding to the FERC without delay. Either way, the results are put into context in the first email to the FERC which states:

> "These emails are the result of about 2 hours of research by me and my staff …"

A later email (the same day) to the FERC states:

> "Good luck with your response to Washington. I hope I provided what was needed …"

Thus, it is evident that very little actual research, if any, was conducted. This was merely an exercise in document gathering. The emails included nine detailed borehole logs from 1962 (including core photos from one hole), summary geological profiles and foundation maps.

> "…it is the best we could do in this short time. If you zoom in, you will see the hole locations better. The holes on the hillside were drilled along a chute alignment that was not built. As you can see, the site was thoroughly explored. Next I will be emailing you several of the actual drill hole logs. They all show that the area of interest is composed of solid, essentially non-erodible bedrock with an insignificant layer of top soil that will erode."

However, as seen in Figure C-13, all of the detailed information in the following emails are from explorations in the crest area of the emergency spillway. Although reference is made to the borehole investigations downstream from the emergency spillway crest structure (along the original service spillway alignment), no detailed information whatsoever was provided for the area downstream of the overflow weir, neither any of the detailed borehole logs (as summarized in Table B-1 above) nor the geological sections prepared in 1962.

> "The attached drawing contains summary drill hole logs for some important drill holes. We could not find the actual drill hole logs, just this sheet."

Thus, the detailed geological descriptions that would have given critical information on strength and jointing (which would have indicated the likelihood of erodibility) downstream from the emergency weir were not accessed. However, the nine borehole logs that were accessed provided ample evidence of poor to very poor rock conditions. Even if one assumed that moderately weathered rock was non-erodable, six of the nine logs show worse than moderately weathered rock conditions. Two logs show strongly weathered rock to depths of 12 feet and 17 feet, and all six logs indicate moderately to strongly weathered, moderately weathered to soft, poor to fair conditions to depths ranging from 22 Feet to 44 feet.

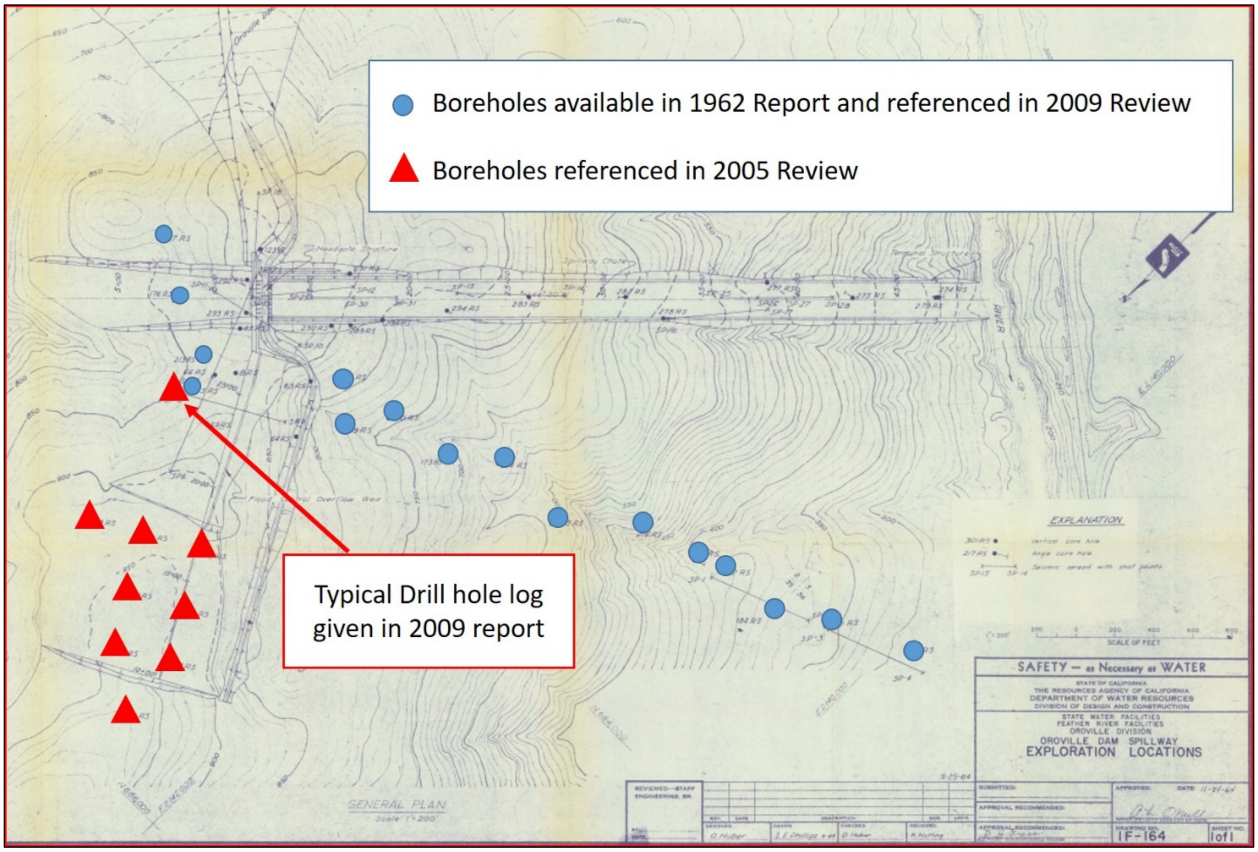



**Figure C-13: Borehole Logs Available for Review**

The email train includes a summary memorandum [C-30] from the Project Geology Section to the Civil Maintenance Branch of DWR. It is stated in the memorandum that:

> "The Emergency Spillway does not empty onto a bare dirt hillside. Instead, it empties onto a hillside composed of solid amphibolite bedrock extending from the spillway crest down to the Feather River. *Where the rock is fresh, it is hard, dense, fine- to medium-grained, greenish-gray to black and generally massive.* Even though this rock contains numerous narrow shears and schistose zones, variable weathering, joints and fractures, it is considered an excellent and competent spillway rock." (italics added for emphasis)

The sentence in italics is a direct quote from the 1960s Interim geological reports, and remains true, but irrelevant and misleading when taken out of context. The IFT could find no basis for the remainder of the claim that *solid* amphibolite extended “from the spillway crest down to the Feather River.” Since no information covering the area from the crest to the river was included in the emails, the IFT assumes that this information was never accessed in the time available, and that no work was done as a check against this claim.

In the ensuing back-and-forth email communication [C-29], DWR notes to FERC:

> “You probably recall that we all discussed this exact issue during the PFMA last year, and my recollection is that we dismissed it. I remember we said that a lot of debris would get washed into the river, but I think we concluded that there would be no structural damage …”

The memorandum [C-30] concludes:

> “Based on the information presented in this Office Memo, as obtained from the reports in the Project Geology files, it is my belief that Emergency Spillway at Oroville Dam is a safe and stable structure founded on bedrock that will not erode.”

Although a reasonable statement for the overflow weir itself, where significant excavation had been made to expose an acceptable foundation surface, it has no relevance in regard to the downstream conditions, where no such excavations had been made to remove areas of unprotected, highly erodible, weathered and jointed rock to significant depths. The major inconsistency between the conditions shown on the nine borehole logs and the sweeping generalization of acceptable rock conditions downstream from the emergency overflow weir was not recognized by either the author, or the FERC recipient of the memorandum.

This memorandum, based on work performed within one day, apparently without DWR peer review, and obviously not critically reviewed by FERC, is referenced in all future project reviews, apparently forming the basis of the ongoing overall misrepresentation of the bedrock conditions at the Oroville Dam spillways from that time forward.

### 3.13 <u>2006 State Water Contractors Review</u>

The Metropolitan Water District of Southern California was working in concert with the State Water Contractors on the Oroville FERC relicensing, and subsequently reviewed the 2005 Motion to Intervene. The results of the review were provide to the FERC on May 26, 2006 [C-31]. The issue regarding impacts of emergency spillway usage are covered in the following:

> “… Sutter County/Yuba City have not offered any evidence to cast doubt on the integrity of the hillside downstream of the emergency spillway …”

> “Friends of the River cites an August 2002 Yuba County Water Agency Technical Memorandum … but the cited memo is nowhere to be found in the record of this proceeding. The Commission should either seek to have the YCWA memo included in the record or to consider the information as anecdotal and without technical merit.”

Apparently, the latter approach was adopted, and the potential for erosion was ignored.

The IFT was approached by the registered civil engineer who in his capacity at the time, was asked to review the Motion to Intervene. He informed the IFT that the purpose of the review was to:

> "…determine whether it included new technical analyses that would lead one to doubt the decisions that drove the design of the Oroville Facilities….. In drafting the SWC/Metropolitan response to FOR's filing, I relied on the assumption that the dam designers used appropriate safety factors in their design and that geologists that oversaw the construction competently inspected the dam foundation preparation to assure that it supported the assumptions used in designing the weirs."

Thus, as with the 2005 DWR review, there was no actual technical questioning by the SWC of the geologic conditions downstream from the emergency spillway weir.

### 3.14 2009 Emergency Spillway Erosion Study

Another review of the conditions downstream of the emergency spillway was undertaken in 2009. This was in response to a concern raised by the FERC in their comments on the Seventh Independent Consultants' Safety Inspection Report. This report had recommended evaluating the effects of deposition of eroded material from the use of the Oroville emergency spillway on the downstream Thermalito Diversion Dam reservoir.

The result is a memorandum from DWR Engineering (Dams and Canals Section) to DWR Operations and Maintenance, dated April 21, 2009 [C-32]. This work estimated the volume of erosion that could be expected downstream along the emergency spillway, and references both the 2005 memo and the original 1962 investigations. However, the work apparently started with the premise of the 2005 memo that the foundation was considered "an excellent and competent spillway rock," and that "only soil and decomposed rock" is erodible. The 2009 Report includes this statement:

> "That memo (DWR 2005) also describes from an engineering geology point of view that there is 'an insignificant amount of top soil' overlying the bedrock. In that context, 'insignificant' means that even if the reservoir water were to flow over the Emergency Spillway, the water would very quickly encounter solid bedrock on the downstream side of the spillway, and as the thin mantle of soil would be eroded away, the rock would remain very resistant to erosion. Typical drilling logs and core samples are shown in Figure 5 and Figure 6."

Interestingly, the one typical log from downstream along the alignment given in the report (173RS) is referenced as coming from the 2005 report, however this is an apparent error – it does not appear in the copies of the 2005 report available to the IFT. That particular borehole log is from the 1962 report, which was obviously researched to a greater extent than in 2005: the 2009 report accurately quotes from the original 1962 text, and also accurately summarizes borehole information in a table documenting depths of "soil and decomposed rock" and depths of "strongly and/or moderately weathered rock," up to 36.5 feet, which is in direct contradiction to the statement quoted above.

The IFT learned through interviews that the DOE was given about 200 hours for this study, with 20 hours allotted to the DOE Project Geology group to look up old reports and to give an opinion on what was erodible. Thus, there was adequate time to properly access the downstream information that was overlooked in 2005. However, the stark disparity between the 1962 information and the basis of the 2005 claim was not recognized, nor apparently reviewed. By quantifying the amount of expected erosion upon use of the emergency spillway, this memorandum effectively sealed the impression that only an average of 4 feet of "soil and decomposed rock" will be erodible and wash into the Feather River if the emergency spillway operates, and this impression is apparently carried forward in later PFMAs and various project reviews without question.

### 3.15 IFT Findings in Subsequent Interviews

The IFT had the opportunity to interview a number of persons involved at the time of the 2005 and 2009 reviews of spillway foundation conditions. The individuals involved with this work are "2nd generation" geologists at DWR, without having had direct involvement with any of the original investigations and design. A number of relevant facts and opinions emerged:

**2009 Erosion Study [C-32]**:

- The principal DOE author of the 2009 Erosion Study depended on all of the previous descriptions and did not question the data in the borehole logs.
- DOE was specifically tasked to look at only the erosion potential of the soil and completely weathered rock. There was never any discussion whether or not strongly weathered rock would erode. There was also no inclination to do additional investigation due to workload and time constraints.
- O&M had a preconceived notion that only this material would erode, based on the 2005 Review by DOE (the same group, but not the same individuals tasked with the 2009 study). O&M commented that they relied on DOE, and would not "go back and second-guess" the DOE Project Geology group.
- The 2009 study was not peer reviewed in detail.

**2005 Review [C-30]**:

- There was no time available to do any assessment or evaluation during the preparation of the 2005 memo: it was essentially an exercise of gathering the series of borehole logs and plans, and writing the memo. From various interviews, it was apparent that there was a great reliance on previous recollections and impressions. The DOE Project Geology group agreed to answer the FERC request within a few hours "because there was a longstanding level of confidence in the rock …"
- Prior to the 2005 review, no one in the Project Geology group had looked at any geologic details. Impressions had been made on geology staff by those who had been there. They had "heard all the stories" from those who had been involved, and had previously relied on these. The following represents this "oral history":
    - The general impression from the original exploratory trenches by bulldozer in the 1950s was that the rock was good quality, sound and firm. In hindsight, interviewees admitted that these trenches would have only been on the order of 3-5 ft. depth, so this interpretation does not seem credible. However, this was the impression at the time.

  - o Following the trenches, a number of seismic refraction surveys were run, showing relatively high velocities in the upper velocity layer which seemed to confirm the initial impressions (however, this is not borne out in review of the 1964 Interim Geology report).
  - o Results from the borings were seen as representing localized conditions, not as a fatal flaw.
    - – These first three points above, based on oral history, are in contrast to the 1964 Report, which as previously noted, clearly states "Since drill hole data are more reliable than seismic data, 'Depth to Sound Rock' as presented in this report is based primarily on drill hole data."
  - o There may have been bias, because all the boreholes were located along a topographic low, with the assumption that rock conditions would be somewhat worse along this alignment than an average across the entire width of the emergency spillway chute.
- There was not believed to be any reason to question, or check this mindset. Collective thinking was that the rock was overall good quality, and that there just was not enough poor quality rock to be a problem.
- Bill Akers (signator to the Final Construction Geology report) moved into DSOD in his later career and was active until 1984; he was highly respected and had a large influence. He never raised any concerns regarding potential erosion.
  - o The IFT can find no evidence that Mr. Akers was involved with either the 1962 or 1964 investigations, nor was he a member of the on-site geology team during construction. Although stationed in Sacramento, one assumed he would have been aware of the major issues and construction claim involved with the service spillway, but would have been on the side arguing that the foundation conditions were fully acceptable. One long-retired employee who knew Mr. Akers remembers much discussion regarding the required foundation cleanup, but not the details. He noted that Mr. Akers used to boast that DWR had as many geologists in the 1960s as the Corps and Bureau combined.
- There was agreement that the relevant issue with respect to erodibility is not necessarily only the degree of weathering, it is joint spacing and condition. The significance of the jointing was not recognized – although simple in hindsight, it is not necessarily apparent in the reports, only the borehole logs.
  - o The full descriptions are only prominent in the detailed borehole logs, and as these were never reviewed by any "2nd Generation" geologist in any detail, the significance of the jointing and other rock mass properties was missed.
- When asked directly by the IFT, no opinion was given as to why any geologist would interpret strongly weathered, strongly fractured, very soft or soft rock with open jointing as being non-erodible.

During its investigation, the IFT found that there still seems to be a reluctance to believe the rock condition was misinterpreted or misrepresented, and there is an apparent willingness to defend the geological interpretation. A number of interviewees were quick to note that the rock held up well to erosion where confined, and that blocks did not necessarily erode until they toppled and shattered. Others expressed disbelief in the possibility of the severity of erosion until they had actually seen it occur, and one opined that "nothing" could have withstood the flow.

## 4.0 SUMMARY

The misconceptions regarding the erodibility of the bedrock along the two spillway alignments are central to the development of the Oroville incident. It is now clearly evident that actual bedrock conditions, and the implications of those conditions, were well documented prior to and during construction, but there was no post-construction recognition of the weathering potential of the rock types present at Oroville. Detailed and accurate information was not properly accessed in subsequent years; rather, inaccurate and incomplete summaries of information were passed on through generations of DWR personnel.

There is reasonably common knowledge in current geological practice that foundations involving the rock types present at Oroville (amphibolite, greenstone, ophiolite) might point toward a particular susceptibility for pronounced weathering, and that the weathered by-products would be vulnerable to erosion. Since the 1980s at least, based simply on regional geological mapping, a qualified engineering geologist should have been able to recognize this potential. However, there apparently was never any attempt to have a new look at the data in order to make that interpretation.

The earliest geological site investigation reports from 1948 clearly recognized this issue, and actually proposed a solution for the emergency spillway that is now being utilized in the remediation works – a deep cutoff to fresh rock. More detailed investigations, reported in 1962, properly and fully described the typical deep weathering pattern in bedrock, and clearly recognized its very irregular pattern, noting that "weathered rock will of course be subject to relatively accelerated erosion; where this is critical, the rock should be protected." The descriptions in the borehole logs are detailed and very precise, and even a cursory inspection of these logs clearly indicates the significant depths of strongly weathered, strongly fractured, soft rock with open jointing, covering a range in quality that was described as "poor to fair" rock conditions. The IFT believes that any even relatively unexperienced geologist or geotechnical engineer today would consider these "poor to fair" conditions as being very erodible, as did the first geologists onsite in 1948. However, this detail is lost in the summary logs and sections reproduced in later reports.

The same degree of care and professionalism is evident in the production of the borehole logs, and geological descriptions during further investigations in 1964. However, there appears to have been a major change in opinion regarding the geotechnical acceptability of moderately weathered rock. The 1964 updated conceptual service spillway chute profile shows a large portion of the chute founded on moderately weathered rock, which was now considered to be an adequate foundation for most of the structures, although it "may require some special treatment." The required special treatment was not defined in the report.

Comments made in 1962 on potential scour were not carried forward to the 1964 report. However, from comments in the 1964 report, the geologists were certainly aware of scouring as a failure mode. It is likely that the geologists were relying on the spillway chute designers to provide adequate protection for the poor foundation, while the designers were relying on the geology to provide a good, solid foundation, as was indicated in IFT interviews.

The 1964 report also recommends an underdrain system to be built upon the invert. Evidence suggests to the IFT that the purpose of the drains was mainly, or solely, to deal with potential seepage from the bedrock. Thus, the same report that very professionally describes the poor to very poor geological conditions at the proposed invert elevation of the service spillway, fails to connect the geological descriptions with the potential for scour of the rippable rock due to flow in the underdrains. Regardless, it is evident that prior to construction, there was never any intention of founding the slabs or placing the underdrains on a strongly weathered rock foundation.

The 1964 report states "… most of the non-rippable rock will be adequate for this structure," inferring that the moderately weathered rock would be non-rippable. Likely based on this assumption, specifications for the spillway chute excavation did not explicitly cover the conditions in which moderately or strongly weathered rock could be easily removed without heavy-duty power excavating equipment beyond the excavation limit lines. A 1965 design review pointed out that "backfill concrete will probably be required for the purpose of filling deeply weathered pockets," but fails to recognize that the specifications do not adequately cover the situation of rippable rock below the grade line of the excavation.

There was a major claim by the Contractor during construction related to the change to the drain design and over-excavation required. DWR clearly documents that they originally took the position that any excavation with heavy power equipment beyond the minimum excavation limit lines would not be payable. However, it was later agreed that payment was due for all shear-zone excavation and the concrete backfill costs for material so excavated by heavy duty power equipment. This is likely one reason why the shear zones were carefully mapped and individually numbered. However, the agreement did *not* include areas of highly weathered rock.

It is clear that portraying foundation conditions at invert level as being acceptable was central to DWR's defense against the Contractor's claim of poor rock conditions. It remains speculation as to whether a DWR organizational reluctance to acknowledge out-of-specification foundation conditions, and instead to consider the encountered rock conditions as being acceptable, became entrenched as part of oral organizational history. The final construction reports simply state "In the chute, there was very little extra excavation directed." The report also either did not recognize, or chose not to comment, on the fact that the anchor tests were not conducted in the worst foundation as stated, but rather the "worst foundation available" at the time of the tests, prior to exposing the moderately to highly weathered areas.

The detailed borehole logs containing the accurate descriptions of the moderately to highly weathered bedrock are given in the 1962 and 1964 Interim Geological reports, but are omitted from the Final Geological Report. The opening description of the Final Geological Report, when read in isolation from the remainder of the report, describes very favorable foundation conditions, and this depiction may be a major factor in the ensuing misinterpretations of bedrock conditions.

In the early to mid-2000s, as part of the Oroville Dam re-licensing process, external groups questioned the safety of the emergency spillway due to the potential for bedrock erosion, based on the original records and reports. DWR was requested by the FERC to investigate this, and a very brief review was undertaken. It was essentially an exercise of pulling together a series of borehole logs and plans, and writing a short cover memorandum. However, all of the detailed information that was supplied to the FERC was from the crest area of the emergency spillway, apparently in support of the claim of acceptable foundation conditions for that structure. Six of the nine borehole logs showed significant depths (22 to 44 ft) of poor to fair rock, moderately to strongly weathered, moderately hard to soft rock conditions. No detailed information whatsoever was provided for the area downstream of the overflow weir. Regardless, the position is taken in a 2005 DWR memorandum that solid amphibolite extended "from the spillway crest down to the Feather River." It seems apparent that there was a great reliance on the recollections and impressions from previous generations of DWR geologists. The major inconsistency between the borehole logs as presented and the claims made in the memorandum was not recognized by either DWR or FERC. To the IFTs knowledge, no work was ever been done, either internally or externally, as a check against these claims.

A study was undertaken in 2009 to quantify the expected erosion upon use of the emergency spillway, specifically to look at the erosion potential of the soil and completely weathered rock. Although the actual detailed geological descriptions were accessed for this work, the stark disparity between these descriptions and the basis of the 2005 claim was not recognized. Neither the 2005 nor the 2009 work was apparently peer reviewed in detail. By quantifying the amount of expected erosion upon use of the emergency spillway, the 2009 work effectively sealed the impression that only an average of 4 feet of "soil and decomposed rock" would be erodible and wash into the Feather River during activation of the emergency spillway, and this impression is apparently carried forward in later PFMAs and various project reviews without question.

Had the recognition of the weathering potential of the rock types present at Oroville been made at any time post-construction, it should have led to (1) questioning if such conditions had been properly recognized and managed during design and construction (e.g. making appropriate adjustments in excavation depths, anchor lengths, or other measures); (2) questioning the erodibility of the foundations, particularly if there is knowledge that weathered materials had been left in place; and (3) questioning whether the slab design was suitable for as-built foundation, given the presence of weak and erodible foundation materials.

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# Appendix D

# Post-Incident Forensic Field Investigations

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## 1.0 BACKGROUND

Appendix D provides a description of the scope and results of the forensic field investigations performed after the February 7, 2017, incident, which were carried out by or under the direction of the Independent Forensic Team (IFT). In addition to descriptions of factual information and observations obtained from the field investigations, comments are made on where departures from original design were noted. Certain basic interpretations are also provided in the narrative of this appendix. However, comprehensive discussions and implications of such information and their relationship to the February 7, 2017 incident and to broader aspects of design and construction practices are drawn out and presented in the main report.

This appendix does not include detailed descriptions, results, or final conclusions of tree root investigations that were carried out by others for DWR. The Main Report provides a summary of these investigations and preliminary conclusions available at the time of report preparation.

## 2.0 FORENSIC FIELD INVESTIGATIONS PROGRAM

After initial site inspection visits by IFT members in April and May, 2017, a plan for proposed forensic field investigations was submitted to DWR on May 17, 2017 [D-1] and then developed into more detailed field instructions [D-2]. A general plan showing the locations where forensic field investigations were ultimately conducted is provided in Figures D-1 and D-2, both of which were adapted from DWR [D-3]. All forensic investigations conducted as part of this exercise were related to the Service Spillway and do not bear on the Emergency Spillway.

Unless indicated otherwise, all photographs shown as figures in this appendix were taken by IFT or DWR personnel working with the IFT.

The following is a summary description of field investigations specifically requested and directed by the IFT.

1. Service spillway lower chute between approximately Sta. 33+00 and Sta. 43+50
    a. Detailed inspections of the remaining and damaged sections of the lower chute. These investigations were conducted by a member of the IFT May 7 through 9, 2017, and on various days between May 24 and June 2, 2017. Activities included examination of locations, characteristics, and condition of remaining concrete works and adjacent areas, herringbone drains, control joints, reinforcement, foundation anchors, and foundations beneath.
2. Service spillway upper chute between Sta. 26+00 and Sta. 28+00
    a. <u>Chute Slabs</u>. Detailed investigations of the spillway concrete chute slab sections in the upper chute area were conducted by a member of the IFT between May 24 and June 2, 2017. These included removal of sections of concrete by saw-cutting (forensic trenches) and use of mechanical breakers at locations designated Areas A, B, C, and D. Activities included examination of locations, characteristics, and condition of spillway slab concrete, chute slab repairs and patching, slab joints, concrete reinforcement, foundation anchors, and characteristics of the foundations beneath. DWR resources

provided valuable assistance in this work. In addition, results of concurrent investigations and studies conducted by DWR were obtained and used by the IFT.

b. Slab Underdrains. Investigations of the herringbone underdrain drain system were carried out by DWR resources, with IFT input. These included removal of sections of concrete by saw-cutting to expose drains, examination of foundations beneath, and remotely operated vehicle (ROV) inspections of sections of drain, where accessible.

c. Collector Pipe Drains. Investigations of the collector pipe drainage system were conducted by DWR resources with IFT oversight. Originally this was to involve excavation of backfill and exposure of selected sections of the collector pipe system on both sides of the spillway, outside of the chute walls, in order to examine their characteristics and condition. These activities were cancelled because useful video inspections had been successfully obtained by ROV in these areas.

d. Concrete coring. An extensive program of invert concrete coring in the spillway upper chute area was completed by DWR. The IFT requested additional concrete core holes in selected areas of interest, with the coring extending into bedrock and with *in situ* testing.

3. Tree Root Investigations. Because tree root penetration into the drainage system had been identified as a possible physical factor contributing to the February 2017 incident, the IFT requested specific investigations involving:

   a. Identification of the locations, species, and size of trees within 100 feet of the spillway chute (both sides). Through consultation with botanical experts, estimation of the root pattern and architecture and the potential travel path for tree roots, taking into account topography and ground conditions (e.g. soil, rock, pervious backfill).

   b. Selection of one or more trees that are the largest/closest to the chute and/or similar to those identified as potential contributors for application of suitable tools/methods to expose the root systems of the trees to identify potential root travel paths and extents.

   c. At select locations near the chute and the potential travel path of tree roots, excavation of trenches behind the chute walls to determine if tree roots extend close to the wall and, if so, to what depths.

   At the time the IFT report was under preparation, tree root investigations were still in progress and therefore information on these investigations is not reported in this appendix. A summary of preliminary findings is presented later in the Main Report and it is understood that at a future date a more complete report would become available through DWR.

DWR resources led by Project Geology of DWR Geotechnical Branch conducted three phases of geologic field investigations between March 28 and June 8, 2017. The objectives of these investigations were to provide foundation assessments to support design of the new spillway works, and to provide field support to the forensic investigation program, as described above.

Results of the IFT investigations conducted by DWR are contained in a draft report submitted to the IFT, dated October 11, 2017, and titled “Results of Oroville Dam Flood Control Outlet Spillway Chute Geologic Exploration and Forensic Study, Project Geology Report No. 20-11-52” [D-3]. Summary descriptions of these studies, their results, and basic interpretations are provided in this appendix.

A commentary on a report by HDR on their study of the service spillway collector drain system is provided later in this appendix in Section 7.

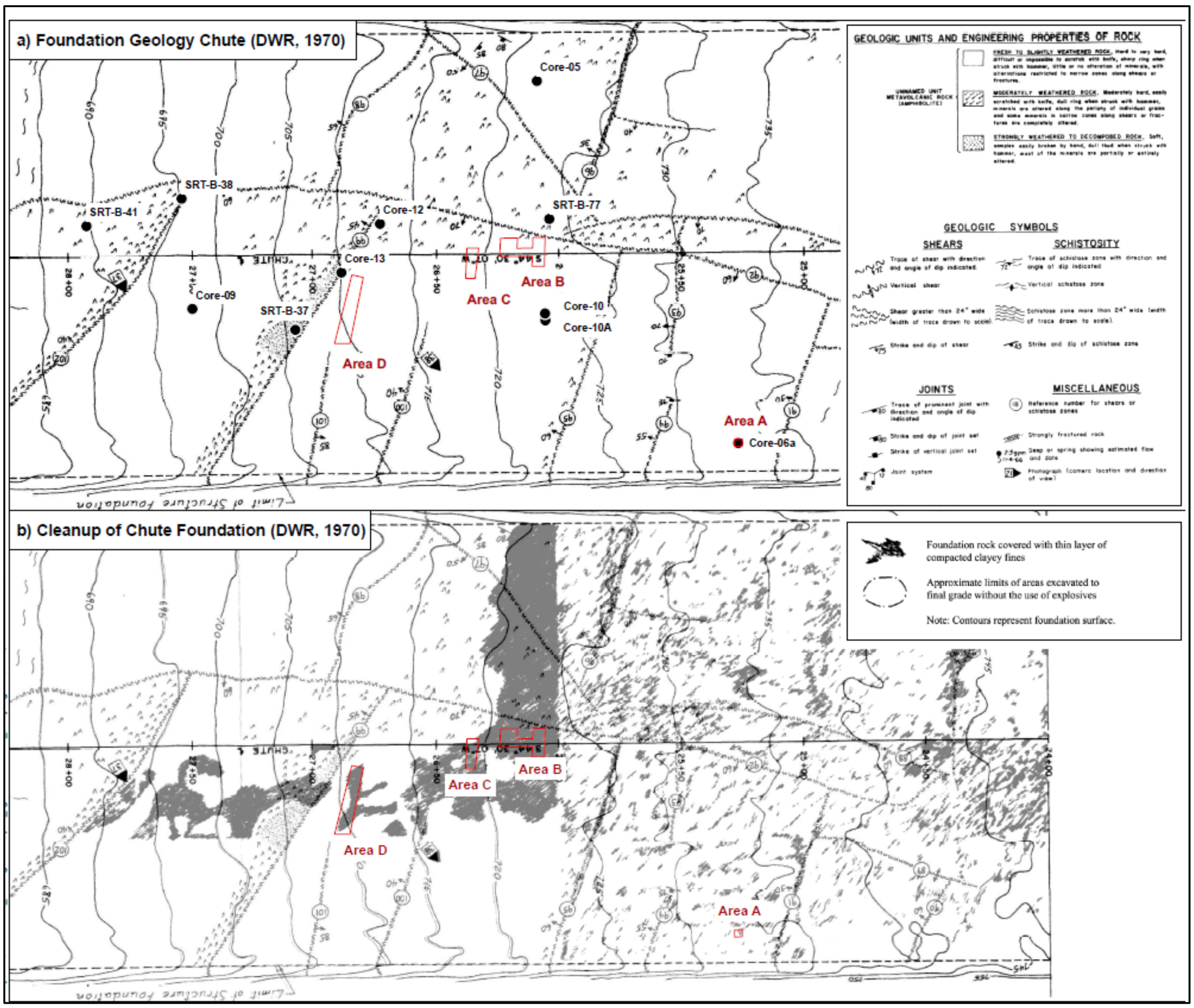



**Figure D-1: Locations of Forensic Trenches at Areas A, B, C, and D superimposed on geologic mapping performed during construction**

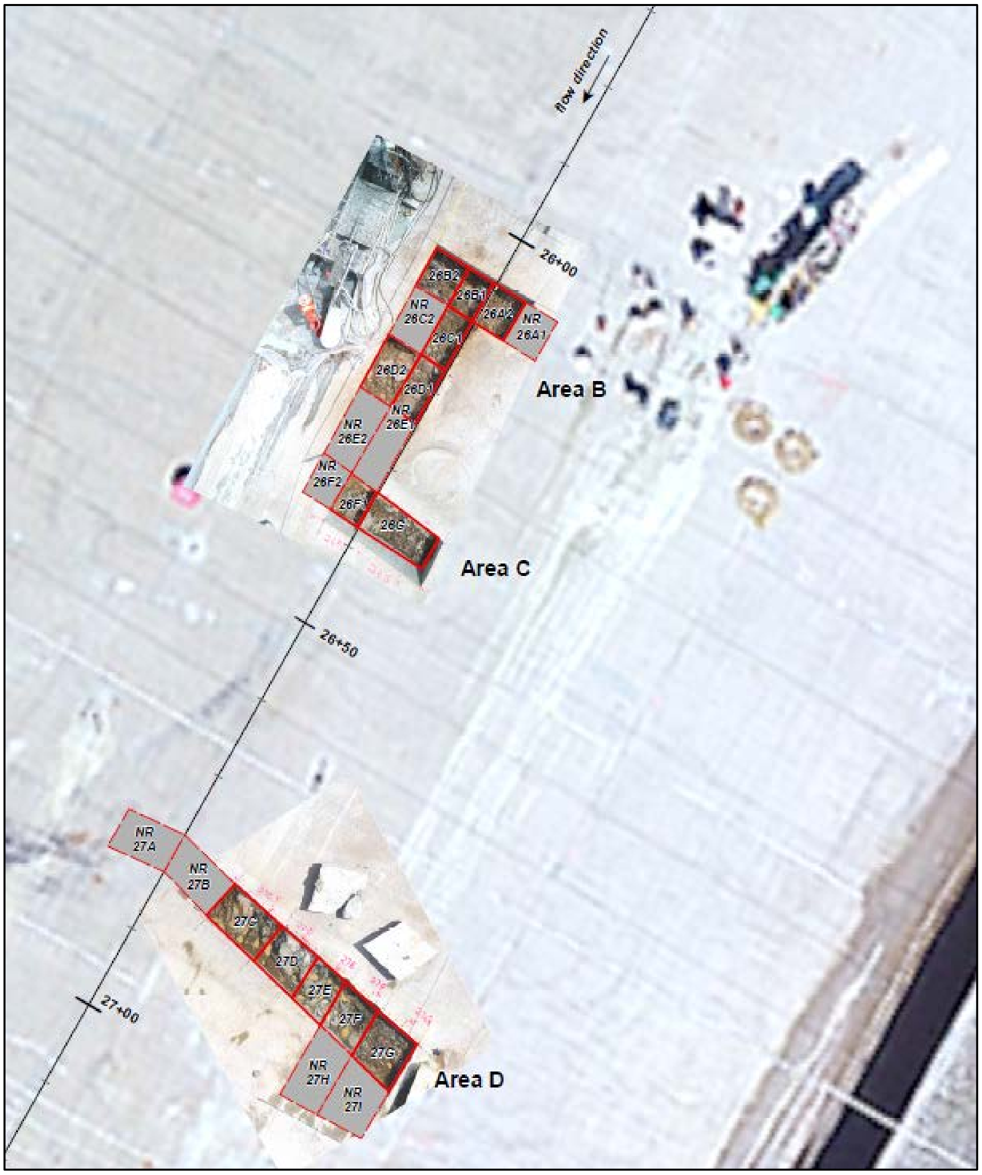



**Figure D-2: Detailed Location of Forensic Trenches at Areas B, C, and D**

## 3.0 SERVICE SPILLWAY CHUTE INVERT SLAB

This section of the appendix provides descriptions of the physical features of the service spillway chute slab as they were observed during the forensic field investigations and concurrent geologic studies by DWR [D-3]. The observations and associated comments relate to various aspects of design and construction, service performance, and current condition.

### 3.1 Foundation Geologic Mapping

The Final Geologic Report on Oroville Dam Spillway [D-4] provides a good record of foundation conditions as encountered in construction of the service spillway chute – see discussion of the

Final Geologic Report in Appendix C of this report. Comments on the bedrock types encountered in the foundation of the spillways are provided in Appendix C.

The construction geology report [D-4] includes foundation geological mapping sheets showing Geologic Units and Engineering Properties, including 124 specific shear zones tabulated along the chute. It also includes detailed drawings showing cleanup of the chute foundation, where the foundation rock was left partly covered with a layer of “compacted clayey fines” (Figure D-3). One objective of the IFT investigation program was to achieve a better understanding and definition of the phrase “compacted clayey fines.”

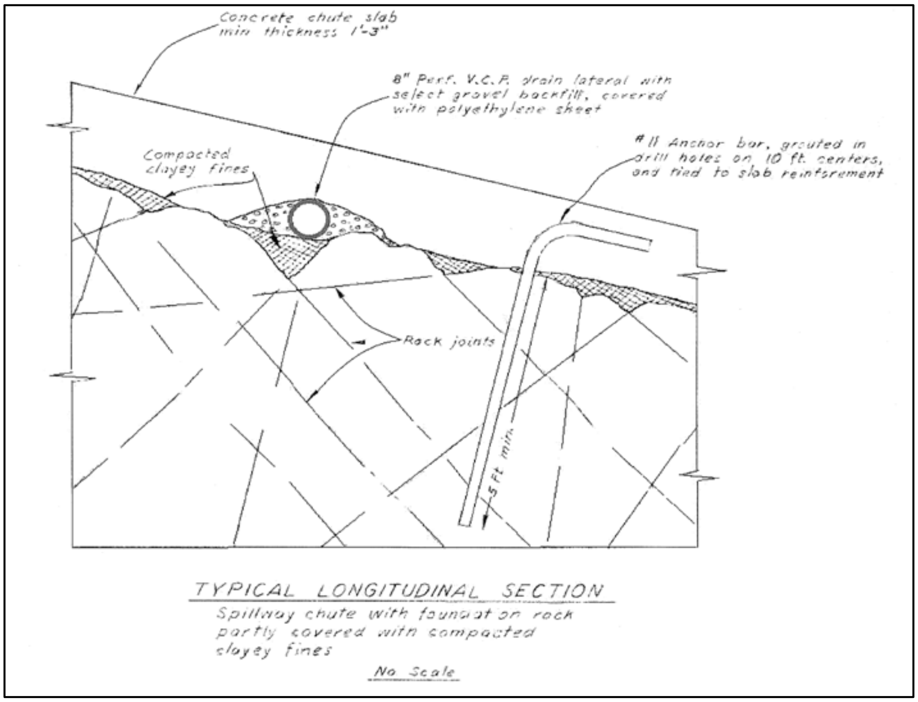



**Figure D-3: Sketch from Construction Geology Report Showing “Compacted Clayey Fines”**

General statements in the report imply favorable foundation conditions for the spillway chute:

> “Foundation rock for the entire spillway is amphibolite, which contains numerous narrow shears and schistose zones. Fresh amphibolite is hard, dense, fine- to medium-grained, greenish gray to black, and generally massive, although a slight foliation (regional structure) is usually present… Joint spacing, within each of the three major joint sets, ranges from a fraction of an inch to several feet, and averages about two feet...”

However, the report also describes:

> “Even though there is only one rock type exposed in the spillway foundation, several factors affect the quality and appearance of this foundation rock. Foremost among these factors is the degree of weathering of the rock at final grade. Other factors which affect the quality of the foundation to a lesser degree are (1) spacing and orientation of joints and shears, (2) thickness, orientation and composition of

> materials in the wider shear and schist zones (3) the presence of strongly fractured or crushed zones (many of which were created by blasting, as shown in Photo 123), and (4) the degree of schistosity or foliation exhibited by the rock."

Details and interpretations of these factors are lacking.

The geologic mapping completed during construction provides documentation of areas where strongly weathered rock was encountered, and described as:

> "Strongly weathered to decomposed rock – soft, samples easily broken by hand, dull thud when struck with hammer, most of the minerals are partially or entirely altered"

The most upstream locations where such conditions were found are documented on the right side of the chute near Sta. 19+00 to 20+50, followed by an area on the left side at Sta. 27+00, with the largest areal extent from Sta. 31+00 to 38+00.

Since the February 2017 incident, DWR geologists have prepared new geologic drawings based on and depicting the original geologic mapping prepared during construction and presented in the Final Geologic Report [D-4]. An extract from the new DWR drawings is presented in Figure D-4. This clearly depicts the locations of some of the weathered conditions in the foundation described in the previous paragraph.

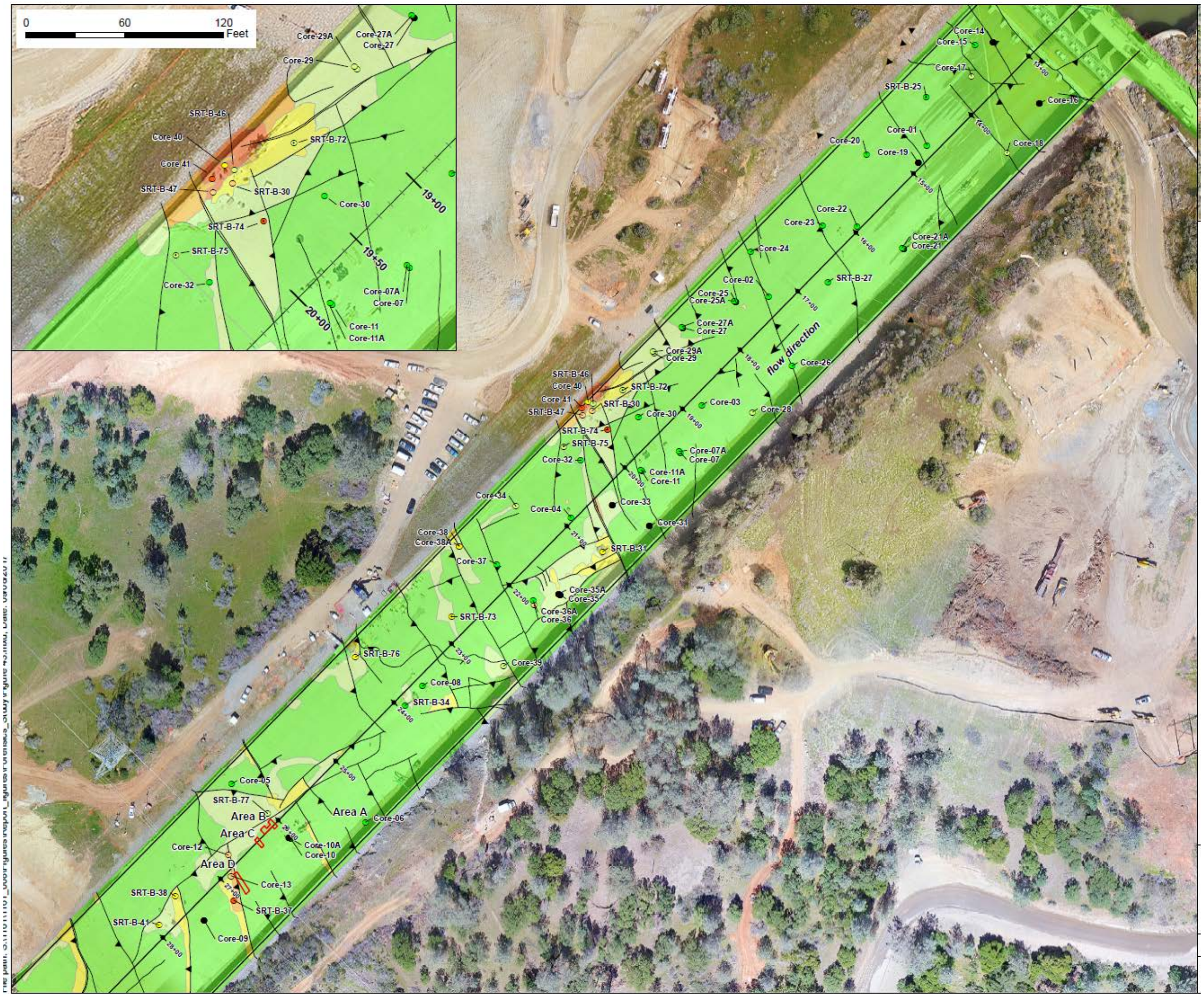


**Figure D-4: Geologic Map of Foundation in Upper Chute Area**

It is noted that the original geologic mapping used only three categories in describing and documenting foundation geologic conditions, as shown in Figure D-5 below.

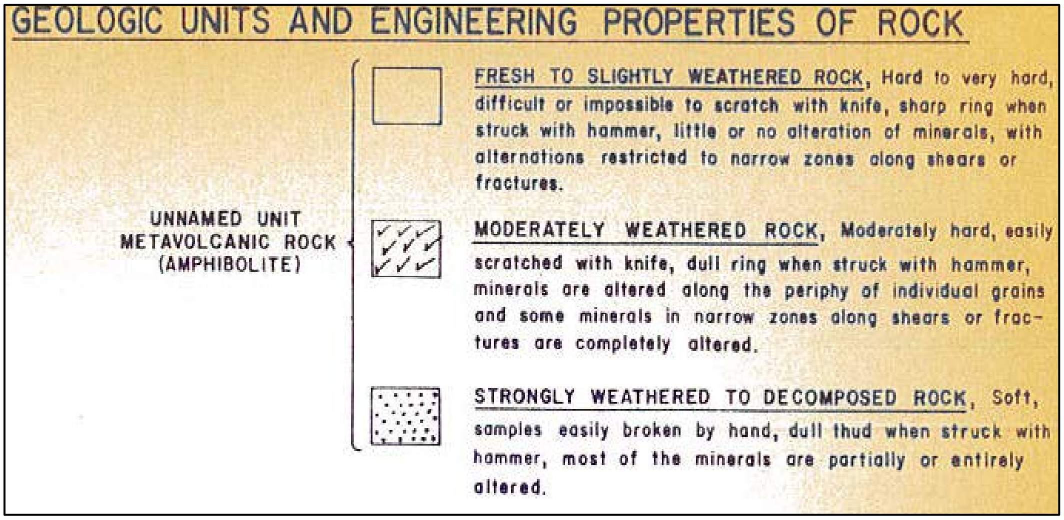



**Figure D-5: Descriptions of Geologic Units Mapped During Construction**

The new DWR geologic drawings show a classification with five categories:

- Slightly weathered to fresh
- Moderately weathered to slightly weathered
- Moderately weathered
- Moderately to strongly weathered
- Strongly weathered

This revision was clearly not based on new information, but was reportedly done to capture certain areas on the original mapping where combinations of symbols were used, e.g. areas with symbols for both "moderately weathered rock" and "'strongly weathered rock," suggesting that geologists at the time were attempting to capture areas that exhibited a transitional nature.

In addition to aiming for a better understanding of "compacted clayey fines," the IFT wanted to examine the foundations in areas where during construction geologists had documented strongly weathered or decomposed rock. For these reasons, investigations included:

- Additional coring in the chute near Sta. 19+00 to 20+50, as well as detailed review of other nearby core drilling conducted by DWR.
- Saw-cutting and removal of sections of chute slabs in the area of Sta. 27+00 in order to expose shear no. 99 and associated weathering (reported during construction as "1.0-1.5 ft clayey gouge and breccia with small pieces of strongly weathered rock").
- Field observations of remaining bedrock in the areas Sta. 31+00 to 38+00. Although deep erosion and complete removal of original foundation grade had taken place due to intermittent spillway operations between February and May 2017, it was still possible to

observe a comprehensive range of rock conditions exposed in areas adjacent to where the spillway chute had once been.

### 3.2 Assessment of Foundation Rock Weathering

From observations made during forensic investigations and results of studies conducted by DWR, the foundation rock types, weathering, and fracture characteristics of the service spillway foundation are generally consistent with conditions documented during original spillway construction.

In only three core holes of the 56 locations evaluated in the upper chute area (Figure D-6) were conditions found to be more highly weathered rock than as documented during original spillway construction. Most of the core holes (39 of 68) were drilled to evaluate foundation rock weathering and to help validate or corroborate the historic geologic mapping conducted during construction [D-4]. In their recent program of investigations, DWR geologists compared results of mapping and sampling conducted as part of 2017 studies with mapping completed during original construction, focusing on characterizations of rock weathering. In general, the rock weathering characteristics observed as part of recent studies correlate well with the historic construction mapping. Similarly, foundation bedrock exposed in the forensic trenches was found to be generally consistent with the original construction geology foundation mapping conducted in those areas.

Because large areas of weathered bedrock (mapped as moderately to intensely weathered and locally decomposed rock) occurred in the foundation of the spillways, it could be asked if weathering could be a condition that changed over the decades since the project was built. It is pointed out, however, that the extent and characteristics of these weathered areas are considered to be approximately the same as they were at the time of construction and as depicted in construction records. Further, it is unlikely that any marked deterioration or rapid weathering could have taken place in four or five decades.

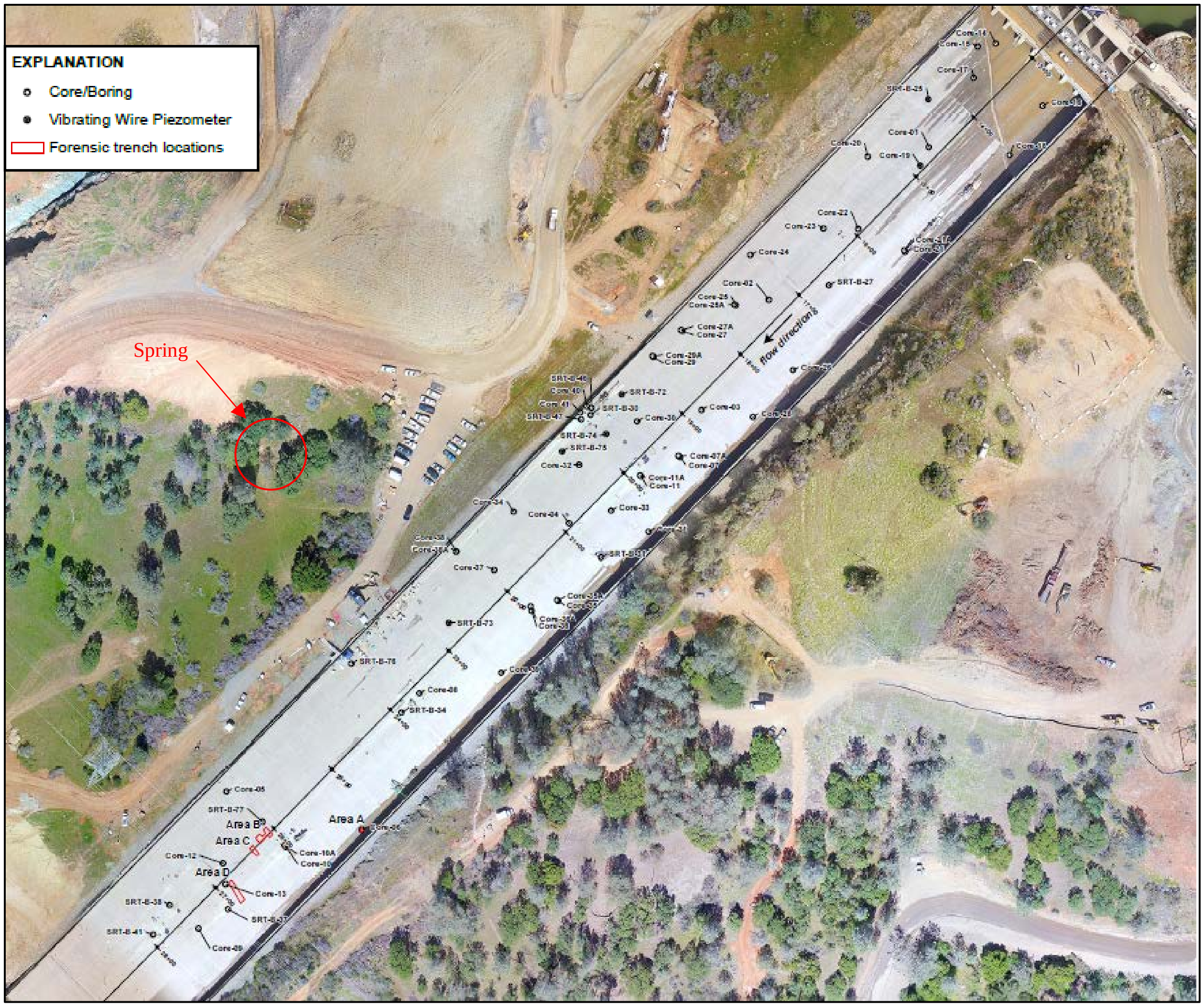



**Figure D-6: Location of DWR Core Holes in Upper Chute Area**

## 3.3 Foundation Preparation and Cleanup

Based on the results of the IFT forensic investigations and other related studies, it is evident that, during original construction, foundation preparation and cleanup were conducted inconsistently and not totally in accordance with the original specifications for construction. In many areas, foundation preparation and cleanup was completed as specified, and concrete can be seen to have been placed directly onto foundation bedrock that was weathered to varying degrees, ranging from fresh or slightly weathered to moderately and strongly weathered, with local pockets of decomposed rock. However, in many other areas, a layer of unconsolidated, soil-like materials was found between the top of the excavated foundation and the bottom of the concrete. It appears that these soil-like materials are the “compacted clayey fines” described in the final construction report. The layer was observed to be very unevenly distributed in presence and thickness. In the forensic trenches (Figures D-7 and D-8) and exploratory core holes it was found to be most commonly ranging from a thin veneer up to 1 to 3 inches in thickness. Local pockets were encountered with a maximum thickness of about 18 inches, which were seen to be typically lodged between pinnacles (saw-teeth) of stronger, less weathered rock in the jagged and rough foundation excavation. Figure D-7 shows the brown-colored "compacted clayey fines distributed between protruding bedrock high-points (photograph from DWR [D3]).

**Figure D-7: Photograph Showing "Compacted Clayey Fines" in Forensic Trench 27C in Area D (approximately Sta. 26+85)**

In Figure D-8, "compacted clayey fines" (tan-brown in color) can be seen immediately adjacent to and below a herringbone drain and gravel pack. The gray-colored fines seen in the photograph are from saw-cutting of concrete.

**Figure D-8: Forensic Trench Area B (26B1, 26B2), Sta. 26+10**

Grab samples of the "compacted clayey fines" were collected from the forensic trenches for gradation and index testing. Results of the classification testing indicate that samples of "compacted clayey fines" range from clayey gravel with sand to silty sand with gravel, i.e. GC to SM in the Unified Soil Classification System [D-5]. The laboratory test results are presented in DWR, 2017 [D-3].

In the field, it was noted that these materials included sharp angular fragments of rock, ranging in size from barely visible to the naked eye up to 1-inch in dimension, suggesting that they are products from excavation of the bedrock foundation, which included blasting. Occasional rounded gravel clasts were observed and are assumed to have originated from spilled drain gravel material as seen in Figure D-8 (forensic trench Area B) and Figure D-9 (bottom of a loose up-rooted concrete block of concrete at the dentates). Figure D-9 shows a cast of a herringbone drain (blue arrows) and evidence of extensive spillage of gravel from drain construction. Also visible in the photograph are loose rock fragments and areas of weathered foundation bedrock adhering to the bottom of the concrete.

Washing of samples during gradation testing confirmed that much of the sand to gravel size fractions consisted of or included angular particles. It is concluded that the "compacted clayey fines" are largely materials left behind after incomplete foundation cleanup. The exact distribution of these soil-like deposits cannot be known with certainty but their presence and thickness obviously varied significantly over short distances.

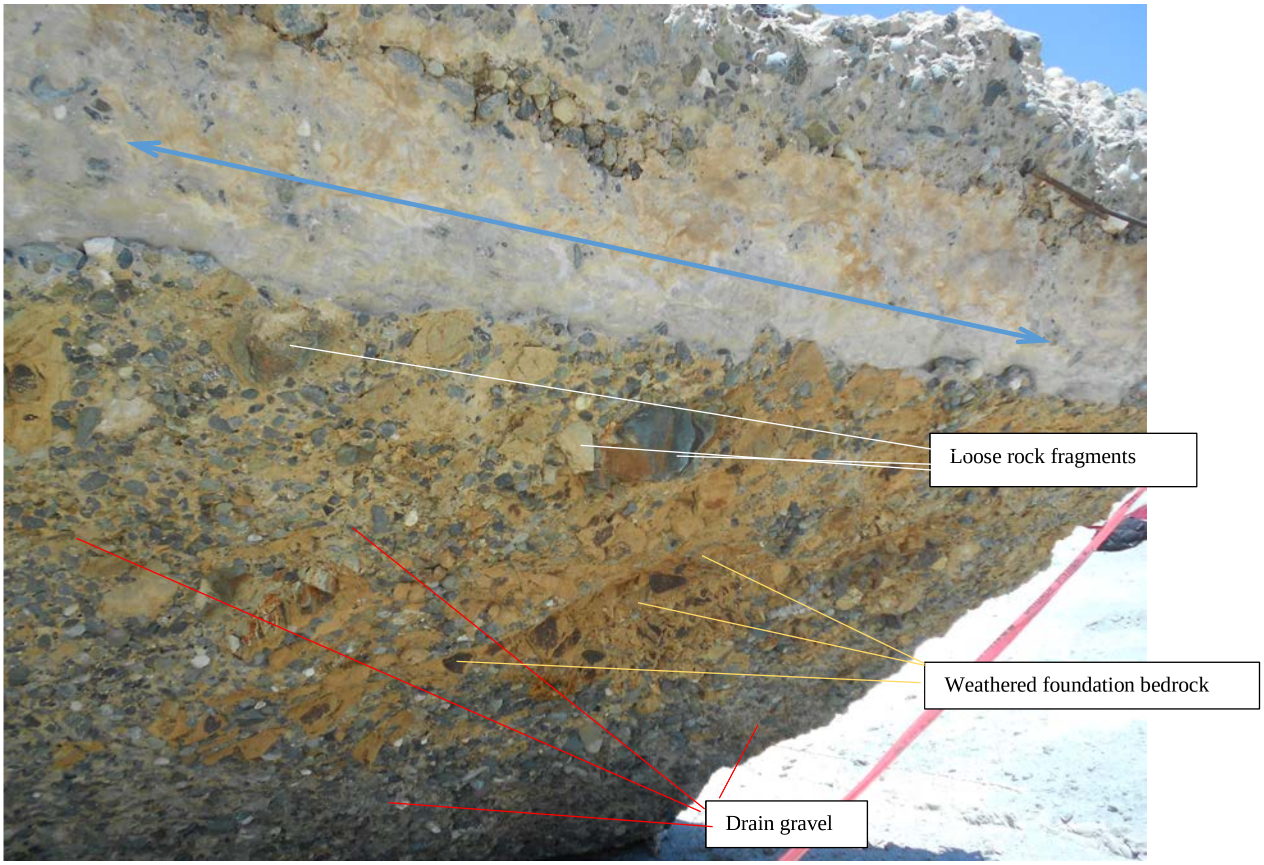



**Figure D-9: Underside of Removed Chute Slab at Dentates**

## 3.4 Clay Mineralogy and Bedrock Petrography

At the request of the IFT, soil and rock samples were collected by DWR from foundation areas of the service spillway for mineralogic and petrographic analysis. The intent was to determine basic physical and classification properties of the materials to evaluate possible origin of the soil materials and to identify any particular characteristics that could potentially have had engineering significance and relevance to the performance of the spillway structures, including the February 7, 2017, incident.

Representative samples included the following types of materials found in the spillway foundations:

- Relatively fresh foundation bedrock, collected from several rock cores and hand samples.
- Clay deposits found in a service spillway underdrain, thought to represent material eroded for an upstream location and washed into the underdrain.
- Compacted clayey fines obtained from beneath the existing spillway concrete slabs, and thought to represent debris from 1966-1967 resulting from blasting and partial cleanup of the spillway excavation at that time.

Information is still pending on requested additional samples representative of weathering products in the strongly weathered rock and decomposed (saprolite) units found *in situ* at the foundations of both the service and emergency spillways.

The results of petrographic analysis and classification of bedrock samples [D6] are in agreement with and confirm the general rock types belonging to the Smartville Complex described for this area [D7, D8]. All samples from rock core are reported to be metamorphosed mafic volcanic rocks and most are likely derived from basaltic or possibly andesitic lithologies prior to metamorphism, although several may have a volcaniclastic origin. Nearly all samples exhibit the characteristic greenschist-facies metamorphic mineral assemblage expected from original mafic lithologies.

Results of X-ray diffraction analysis of the clay materials indicate that both of the subject samples exhibit mineral assemblages generally consistent with greenschist metamorphic facies rocks. Both samples include a combination of sand-size rock fragments and grains of singular minerals. The majority of the rock fragments observed in both samples consist of particles of greenschist and greenstone. It is therefore concluded that the samples originate from the foundation bedrock units in the project site and were not derived from a different provenance or transported from off-site.

The physical properties of the fines content from both samples are shown below.

| **Sample** | **Atterberg Limits** (Liquid limit, LL; Plasticity index, PI) | **USCS[D5]** | **Description** |
|---|---|---|---|
| Clay from underdrain | 33 10 | CL | Lean clay |
| Compacted clayey fines | 31 11 | GC | Clayey gravel with sand |

The IFT is not aware if any standard dispersivity tests have yet been conducted on these materials to help determine their susceptibility to internal erosion or piping.

The clay group minerals identified by the X-ray diffraction analyses in both samples are primarily mixed layer illite/smectite, with expandable (smectite) layers comprising an estimated 85 to 90 percent. In addition to chlorite, minor to trace amounts of undifferentiated illite/mica were also identified. In soils engineering, it is well-known that materials containing clays of this nature can typically exhibit various degrees of shrinkage and swelling (expansion) due to changes in soil moisture, characteristically shrinking upon loss of moisture and swelling upon reabsorption of moisture.

During the course of IFT investigations, consideration was made of the long-term consequences of the presence of smectite clays with expansive potential in the foundations of the spillways. Considering the potential long-term effects of seasonal and diurnal fluctuation in climate, including wetting-drying cycles and effects of recent droughts and climate change in California, the IFT considers it relevant to examine if there could potentially be any unfavorable consequences impacting soil materials in the spillway foundation, either in the compacted clays fines or in strongly weathered/decomposed rock. In particular, it is important to attempt identification of any factor with a progressive nature and which could have contributed to a progressive long-term deterioration of the safe performance of the service spillway.

Three potential impacts, in particular, are pointed out and discussed below:

1. Loss of strength of clay-bearing foundation materials resulting in reduced performance of foundation.
   - Various researchers have reported that wetting and drying cycles can reduce the shear strength of expansive clays and a significant reduction in the effective stress cohesion intercept. Potential consequences of this could include reduced shear strength of weathered foundations and reduced capacity of anchors. It is difficult to evaluate if this has occurred at Oroville given the paucity of currently available data.
   - Experimental data have also indicated that upon repeated wetting and drying, clay soils show decreased swelling ability after every cycle. It is also noted that the first cycle typically results in the most reduction in swelling potential. As the number of cycles increases, additional reduction can be observed until an equilibrium state is reached. Thus in the case of the Oroville service spillway, it is concluded that changes in strength of clay-bearing foundation materials would have taken place early in the project life-cycle and that a significant long-term deterioration in clay strength probably does not constitute a primary contributing factor to the February 7 incident.
2. Loss of inter-particle strength that could lead to increase in vulnerability to erosion by moving water.
   - Soils containing expansive clay minerals are noted to exhibit structural changes upon repeated wetting-drying cycles. Some researchers report washing away of fines from the surface of samples as a result of cyclic wetting of the sample. These effects are mostly related to changes in the crumb structure of soils and could possibly result in an increased susceptibility to erosion. Parker and Jenne [D-15] found that "…piping is facilitated in soils where the materials contain at least 20% smectite clays. Swelling of the smectite tends to disrupt and disaggregate the sediment or soil. In addition to cracking when dry, the swelling clays may become slick, plastic, dispersed, and non-cohesive when wet. This makes them especially vulnerable to erosion by moving water."
   - However, as noted above with the impact of cyclic wetting-drying on clay strength, researchers indicate that an equilibrium is reached within a few wetting-drying cycles and therefore the condition would have been present soon after the project went into operation, if not before. In other words, this would not be a factor that shows any significant progressive change over the life-time of the project. However, the IFT takes note of the fact that soils with a relatively high smectite (swelling clay) content can have an important impact on susceptibility to piping and internal erosion.
3. Changes in the permeability characteristics of clay-bearing materials in the foundations could have taken place by formation of desiccation (shrinkage) cracks which can provide passage for infiltrating water and opportunity for piping by internal erosion. Generally, clay-rich soils exhibit and are known for their low permeability and this appears to be the case at Oroville. While susceptible to erosion, these units are generally stable in their

natural, undisturbed state, but can quickly erode if disturbed or if drainage conditions change in an uncontrolled manner. It is not known if smectite-rich clay units at Oroville did in fact result in formation of desiccation cracks in areas of concern in the spillway foundations, and, if it had, when this would have occurred, whether early in the project history or relatively recently.

Natural soils developed from weathering of smectite-rich rock types can typically exhibit morphological characteristics upon wetting and drying, including desiccation cracks and popcorn texture at the surface of the soils. Popcorn texture is the result of repeated shrink/swell cycles, producing marble-sized pellets. The IFT has not seen such characteristics in natural exposures at Oroville and is not aware if others might have observed them at Oroville. This suggests that either soils with high smectite content are not widespread, i.e. less common than the small number of analyzed samples indicate, or that physical conditions have not yet permitted them to form such characteristic structures.

The IFT points out that there is insufficient information and analytical data on this topic to reach definitive conclusions as to whether occurrence of expansive clay minerals that become particularly susceptible to erosion could have been a factor in the February 7, 2017, incident. There is also insufficient data to evaluate if such processes could have been exacerbated by prolonged drought cycles in this part of California. To investigate any further would have gone beyond the scope of the IFT. At this time, the IFT concludes that this could be identified as a potential but low probability factor with respect to the February 7, 2017, incident. Nevertheless, the IFT takes note of the fact that soils with a relatively high smectite (swelling clay) content can have an important impact on susceptibility to piping and internal erosion.

### 3.5 Groundwater

The underdrain system installed at the service spillway was evidently seen as required to control potential flow and pressures from groundwater seeping from bedrock under the chute slab concrete. This would either be natural groundwater recharge or reservoir seepage passing through, beneath, or around the headworks grout curtain.

During construction some natural groundwater occurrences were noted [D-4]:

> "During excavation of the spillway chute, a few such springs were encountered (Plate 2), but none created a problem during construction, because the flows were very small (2 to 6 gpm). The chute slab underdrains should prove adequate to handle any future flow from these springs."

However, no detailed study of hydrogeology in the service spillway area was made. Since the project went into operation, it appears that no comprehensive evaluation has subsequently been made of foundation hydrogeology, including determination and evaluation of factors such as distinction of groundwater units, groundwater flow patterns, recharge aspects, unit permeability, and transmissivity.

Given the generally low hydraulic conductivity of bedrock units of this type and condition and the geologic structure of the bedrock in the foundation, it is reasonable to expect that groundwater flows would be low to moderate and probably skewed mostly into the low range. Locally higher flows could occur due to presence of occasional open joints, but these would be neither consistent nor prevalent. In shear zones and other areas with higher fracture density, it is probable that weathering products would have reduced fracture porosity. Therefore, the IFT judges that the capacity of the drainage system well-exceeded the yield capacity of the foundation bedrock. The bedrock units, even with occasional higher capacity open joints, would not have had the ability to store and transmit a large quantity of groundwater.

Enquiries were made by the IFT as to whether any springs or other evidence of groundwater discharge had been observed in the area of the spillways since the project has been in operation prior to the February 2017 incident. None were reported. However, on February 15, 2017, a flowing spring was reported located about 250 feet from the right side of the chute at about Sta. 22+00 and indicated on Figure D-6 [D-9].

From IFT interviews with site personnel, it was revealed that this spring was ephemeral and, although it had been observed previously, it had not been a subject of special or continuous monitoring. It was reported that flow was observed only after pronounced rainfall and that it did not appear to have any known relationship to spillway discharge or reservoir level. During forensic investigations in May, an IFT member visited the spring location. It was dry and evidently there had not been any flow for many days, if not weeks. Although there had been several light rain showers, there had not been any significant or sustained precipitation for about three weeks. From field evidence, it appears likely that groundwater discharge had been periodically occurring at this location for some time, perhaps decades. The spring is at the top of a small drainage, which seems to be evident on pre-construction topography and which, in terms of groundwater flow, could be connected to the pervious backfill behind the chute wall on the right side of the spillway.

At this time, there is insufficient information to reach definite conclusions on the true nature and origin of the spring, and therefore the IFT agrees with the DWR decision to conduct routine monitoring of the spring. Various interpretations of the spring need to be explored, including:

- Relationship to reservoir water levels, to precipitation in the site area, to spillway flows, and to groundwater water levels in the vicinity, now being monitored with piezometers.
- In addition, a potential communication might have existed between the spring and the coarse granular backfill behind the right side chute wall. This would have behaved as a collector and able to drain recharge from surface runoff, as well as leakage from collector drains located within the backfill when the chute is operational.

Falling Head Permeability Tests. DWR carried out falling head permeability tests in 19 borehole locations in 2017 investigations [D-3]. The field tests document water takes (i.e. water consumption values, and *not* permeability units) ranging from ≤0.01 gpm at Core-07A to a maximum of about 13.6 gpm at Core-31, and eight locations showed no appreciable water take (≤0.02 gpm). The low values at these locations suggest a low-permeability concrete-foundation interface and tight geologic discontinuities in the foundation rock.

The locations of these tests vary from about 7.7 to 15.8 feet from the nearest herringbone drain and span a wide range of concrete-foundation interface conditions, including: (a) concrete directly overlying rock, (b) concrete overlying clayey fines over rock, and (c) concrete with voids in debris and drain rock. No obvious or direct correlation can be made between distance to nearest drain or to foundation condition. It is possible that local higher water takes (e.g. 13.6 gpm at Core-31) could be related to spill-over of gravel from construction of nearby underdrain(s), possibly combined with the effects of left-over construction debris that not been properly removed. Such instances were observed at several locations during the forensic investigations suggesting the potential for erosion and development of local open networks beneath the concrete slabs – discussed below in 3.6 of this Appendix.

Piezometers installed by DWR. Four vibrating wire piezometers were installed post-incident beneath the chute invert slabs in the upper chute area. Previously no piezometers were installed to monitor groundwater pressures or response to changing conditions (e.g. spillway operations). Data on piezometric (groundwater) pressures are presented in DWR (2017). Since their operation, potentiomentric levels recorded in these piezometers are typically within 12 feet beneath the service spillway chute. During spillway operations in May, groundwater levels were above the concrete-foundation rock interface elevations in all new piezometers, including one (SRT-B-72) which was installed at a depth of about 50 feet below the invert. Groundwater monitored by the instruments indicated a strong correlation with spillway operations and to reservoir pool elevations during the period of May through September 2017.

All of the piezometers showed a near instantaneous response after the spillway gates were opened on May 10, 2017. Water levels represented by the piezometer readings responded readily to changes in spillway flow rates. This, coupled with the observations that groundwater levels were above the concrete-foundation interface, suggests that there is a direct correlation between spillway operation and measured pore pressures. This would support the interpretation that water flowing across the chute penetrated poorly sealed joints or cracks in the chute slabs and infiltrated into the foundation. There is no evidence of generally high piezometric levels in the hillside that could have produced substantial uplift pressures on the spillway slab when the spillway was not in use. Once spillway flows ceased on May 19, 2017, the reduction in groundwater levels appears to correlate with the slow decrease in reservoir levels over the summer months.

Evidence of water flow beneath chute slabs. Although pre-existing large voids at the concrete/ foundation interface were not indisputably observed during the IFT investigations or studies by DWR [D-3], evidence was seen of water flowing under the chute slabs. During the forensic observations, an observation was made of water discharging from a contraction joint at Sta. 23+00 in the upper chute area. This is shown in Figure D-10, where water can be seen welling out of the joint and flowing down the chute surface.

**Figure D-10: Water Discharging from Lateral Contraction Joint at Sta 23+00**

The origin of the water shown in Figure D-10 is not known but is thought most likely to be from leakage through the gate seals which had been channeled (with sandbags) over to the left side of the chute. Some of this water flowed into cracks and joints in the chute slabs where it could then be intercepted by the underdrain system. In this case, it is thought that the water had migrated along the concrete-foundation interface and then had re-emerged at the surface along a joint once it achieved sufficient head and downstream flow was occluded. Another possibility is that this could be artesian groundwater (or reservoir water). These interpretations are considered less likely since similar conditions were not observed in any of the core holes drilled in the area in 2017. However, they cannot be discounted at this time and until a comprehensive groundwater evaluation is conducted (which is outside the scope of the IFT investigations).

### 3.6 Chute Slab Concrete Thickness

Data on the thickness of the concrete chute slabs were obtained by the IFT from:

- Drilling conducted by DWR in 2017 to investigate concrete thickness, quality, and general condition, and to explore foundation conditions. Concrete thickness data from the results of drilling and coring are considered the most accurate and reliable.
- Observations made where the slab thickness was exposed at the margins of damaged areas, in areas where demolition was taking place, and in excavations made specifically for forensic investigations. Concrete thickness was measured by tape where it was safe and access was possible (considered accurate and reliable data). In other areas, concrete thickness was estimated from drone footage and/or from photographs using objects with

known dimensions in the imagery for scale (considered less accurate than data from core drilling or from direct measurement).

- Original construction records of volumes of concrete placed. This source of information is thought to be the least reliable, in particular because of poor correlation with concrete core hole data collected at the same locations.

As part of this study of the chute slab, measurements and estimates of concrete thickness excluded locations within 10 feet of the chute walls because proximity to wall footings could result in greater thickness values.

The minimum thickness specified for the concrete slab is 15 inches (1.25 feet) according to Detail B, Sheet 50, Specification Number 65-09 [D-10].

### 3.7 Upper Chute Area

Data from Drilling. As part of investigation programs conducted following the February 2017 incident, numerous core drill holes were completed in the Upper Chute area, between Sta. 13+00 and Sta. 28+00. From these investigations, the range of thickness of the original concrete invert slabs was evaluated (DWR, 2017).

- From Sta. 13+00 to 18+00, the average thickness was 37.2 inches (minimum 22.8 inches, maximum 62.4 inches).
- From Sta. 18+00 to 24+00, the average thickness was 49.5 inches (minimum 15.6 inches, maximum 81.6 inches).
- From Sta. 24+00 to 28+00, the average thickness was 31.4 inches (minimum 18.0 inches, maximum 50.4 inches).

Some thicker sections of concrete are reported by DWR [D-3] but these values represent locations close to the chute walls on the right side where concrete placement was designed to be thicker than the rest of the chute slabs.

The thinnest concrete is slightly more than the specified minimum thickness of 15 inches. However, none of the data from core holes reflect the thinning of concrete over herringbone drains, see Appendix A.

Data from Forensic Trenches and Temporary Exposures during Demolition. Additional information on slab thickness was obtained from the forensic trench investigations in the Upper Chute Area. Observed concrete thicknesses in the saw-cut trenches ranged from 14.5 inches (1.2 feet) up to 26 inches (2.1 feet), excluding the noted thinning of concrete above the herringbone drains. Concrete thickness over herringbone drains ranged from 7 inches (0.6 feet) to more than 18 inches (1.5 feet). Concrete thicknesses along a centerline exposure between Sta. 25+80 and Sta. 28+00 ranged between 15 to 54 inches (1.3 feet and 4.5 feet).

A transverse (lateral) exposure of the concrete section overlying bedrock was available during demolition at Sta. 20+30. Concrete thickness at this location was observed to range from 25 to 93 inches (2.1 to 7.7 feet) [D-3].

Discussion of Upper Chute Concrete Thickness. Based on the borehole data and observations made in areas of demolition and forensic excavation, it is concluded that most of the upper chute area has a slab thickness significantly greater than the design minimum. Areas with significantly thicker concrete (> 36 inches) appear to correspond approximately to zones where, during construction, excavation depths were increased, presumably to remove unacceptable material, i.e. unfavorably weathered or fractured rock as indicated by geologic mapping. Examples include the area between Sta. 19+10 and Sta. 20+20 on the right side and at the lateral exposure at Sta. 20+30, where zones of moderately weathered rock and smaller areas of strongly weathered to decomposed rock had been recorded on geologic mapping made during construction. The chute concrete in these areas is much thicker than anywhere else (72 inches to > 84 inches), suggesting that excavation was carried down to a depth where foundation conditions were considered of satisfactory quality. This would have been in accordance with the design specifications and suggests that the grade was made up with additional concrete backfill and not soil-like backfill or construction spoil. From core hole data at other locations in the upper chute area, any relationship between variations in concrete thickness and foundation quality is less noticeable and more speculative in interpretation. Other factors could have come into play during construction including variations in excavation methodology, blasting practices, etc.

### 3.7.1 Lower Chute

The chute concrete thicknesses in the area that originally failed in February 2017 cannot be determined with any certainty because, during the chute failure and subsequent spillway flows, the invert slabs were totally removed between Sta. 30+00 and Sta. 33+20 and more than 65% removed from Sta. 33+20 to Sta. 38+50. No concrete coring was conducted in the lower chute area.

The only information obtained on chute slab thicknesses in the lower chute area is from observations of the exposed edges of slabs that remained, though damaged, after spillway operations following the February 7, 2017 initial chute failure. Observations were also made in areas undergoing demolition, after the incident.

Concrete thickness measurements were also taken at a few portions of chute slab that had been displaced, i.e. not *in situ* but transported by spillway flows. These include the large pieces of the slab lying against the dentates seen in several photographs of the damaged lower chute area. From examination of drag-marks and shapes, the possible origin of these pieces has been interpreted, but there still remains some uncertainty as to their exact original locations, and, therefore, these thickness values are not included in our evaluation.

Two zones of the lower chute were a particular focus of attention by the IFT. They are located between Sta. 31+50 and Sta. 34+50 and between Sta. 37+00 and Sta. 39+00. Both areas are known to have been underlain by foundation bedrock that ranged from moderately weathered to strongly weathered, with smaller pockets of decomposed bedrock. The IFT's attention was focused in particular on these areas.

Sta. 31+50 to Sta. 34+50. This area includes the location of the initial spillway chute failure on February 7, 2017, shown in Figure D-11 below. Yellow arrows shown on this photograph indicate sections along which estimates were made of chute slab concrete thickness. Weathered rock is evident by brown discoloration in contrast to relatively unweathered and fresher rock which is green-gray to blue-gray in color. At the time of the forensic investigation, virtually all concrete slab sections had been removed from this area. However, from detailed examination of enlarged photographs and drone footage of the initial chute damage, it was possible to estimate approximate concrete thickness along the exposed edge at Sta. 33+00. The exercise was somewhat impaired because water was still flowing over the edges, and there was a general absence of objects that could be used for scale. Features used to assist in scaling included spacing of dowels in joints (typically consistently spaced at 18 inches), spacing of steel reinforcement (usually consistently at 12 inches each way), and details of contraction joint keys. In spite of these constraints, concrete thickness estimates were made at various locations along sections indicated by yellow arrows on Figure D-11.

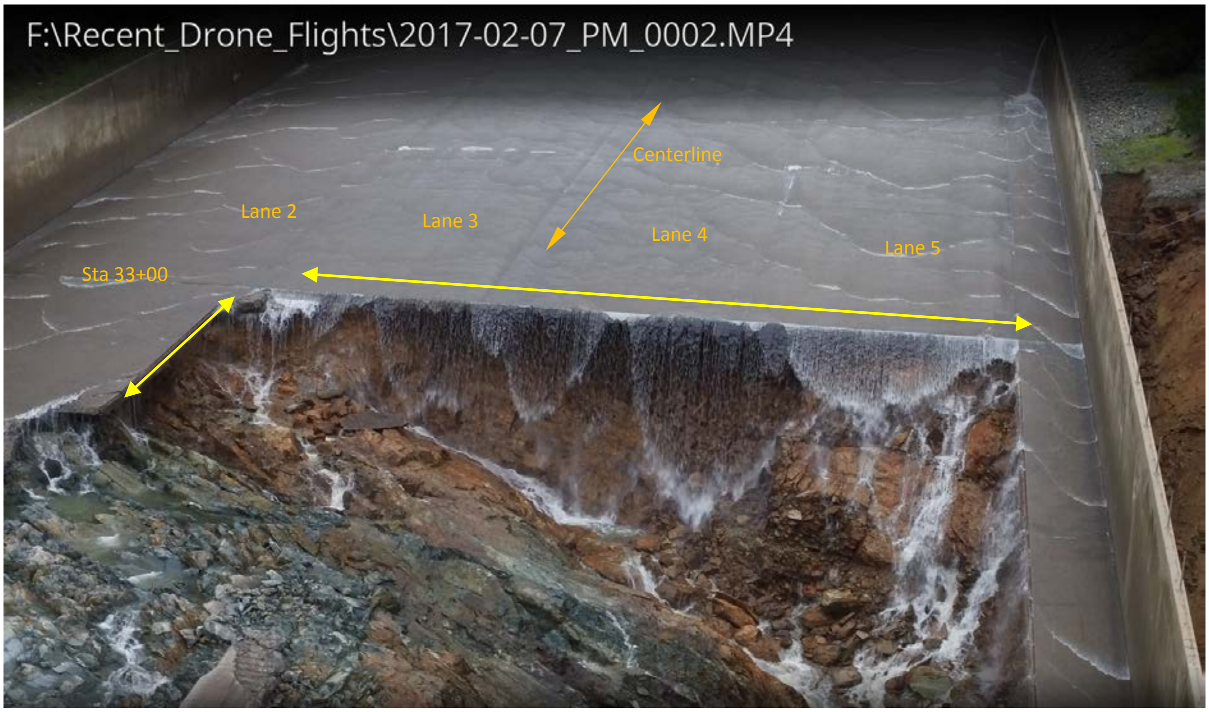



**Figure D-11: Image from Drone Flight Showing Location of Initial Chute Failure Downstream from Sta. 33+00**

Also shown on Figure D-11 are the 40-ft-wide strips of concrete panels that constitute most of the floor of the spillway chute, marked as Lanes 2, 3, 4, and 5. Lanes 1 and 2, each only 9-ft-wide, were placed as part of the footings for the chute walls and were thicker than the rest of the invert slabs. These were not included in the concrete thickness estimations.

Along the Sta. 33+00 line, no exposure was available to estimate concrete thickness in Lane 2. In Lane 3, concrete thickness estimates range from 42 to 50 inches (3.5 to 4.2 ft.) closest to Lane 2, thinning to 36 to 40 inches (3.0 to 3.3 ft.) towards the centerline (between Lanes 3 and 4). In Lane 4, the concrete was estimated to be 24 to 30 inches closest to the centerline, thickening slightly to 38 to 33 inches (2.3 to 2.8 ft.) towards Lane 5. In Lane 5, the slabs were estimated to be about 36 inches (3.0 ft.) thick closest to Lane 4 and thickening to 40 to 46 inches (3.3 to 3.8 ft.) on the left side close to Lane 6. Granted these values are rough estimates. However, they do indicate that the slabs were thinner in the central area, about half the thicknesses on the left or right sides. Moreover, the line at Sta. 33+00 overlies bedrock that was mapped during construction as variably weathered, ranging from moderately weathered rock to materials described as strongly weathered and even decomposed. It does not appear that any concerted attempt was made to adjust depth of foundation excavation (and increase concrete thickness) in response to presence of unfavorable foundation materials that did not meet requirements of the specifications.

Sta. 37+00 to Sta. 39+00. In the more downstream zone, nearly all invert concrete had been removed except for an area some 40 to 80 ft. in width (mostly Lane 2) remaining along the right side of the chute. This area, along with the chute wall and footing, were bridging across a deep channel eroded into the underlying weathered foundation.

A general view of the area and the erosional feature is provided in the photograph shown on Figure D-12. This illustrates apparently thinner invert concrete in an area with weaker, weathered foundation, as documented in geologic mapping during construction and as observed during IFT investigations. In contrast, the concrete appears to be much thicker at Sta. 37+00 overlying more competent bedrock.

**Figure D-12: Photograph of Erosional Feature Between Sta. 37+00 and Sta. 38+50 on Left Side of Lower Chute**

Evaluation of concrete thickness in this area was made by taking measurements with a tape measure at exposed edges wherever possible, or by estimating thickness from photographs taken by an IFT member or from drone-captured imagery. Results are shown on Figure D-12. Data are only shown for areas on the right side of the spillway because exposed edges on the left side (Lanes 4 and 5) are not properly representative of slab thicknesses due to the fact that they had broken mostly at herringbone drains (i.e. at locations where concrete cover was generally much less than the specified 15-inch minimum thickness).

The edge of the Lane 2 panel was well exposed at the chute centerline between Sta. 38+50 (+/-) and Sta. 39+00. The thickness of the concrete in this area ranged from 18 inches (1.5 ft.) to 39 inches (3.25 ft.). The concrete cover over a herringbone drain was 9 inches (observed minimum cover at herringbone drains indicated by * symbol in front of the numeral on Figure D-13).

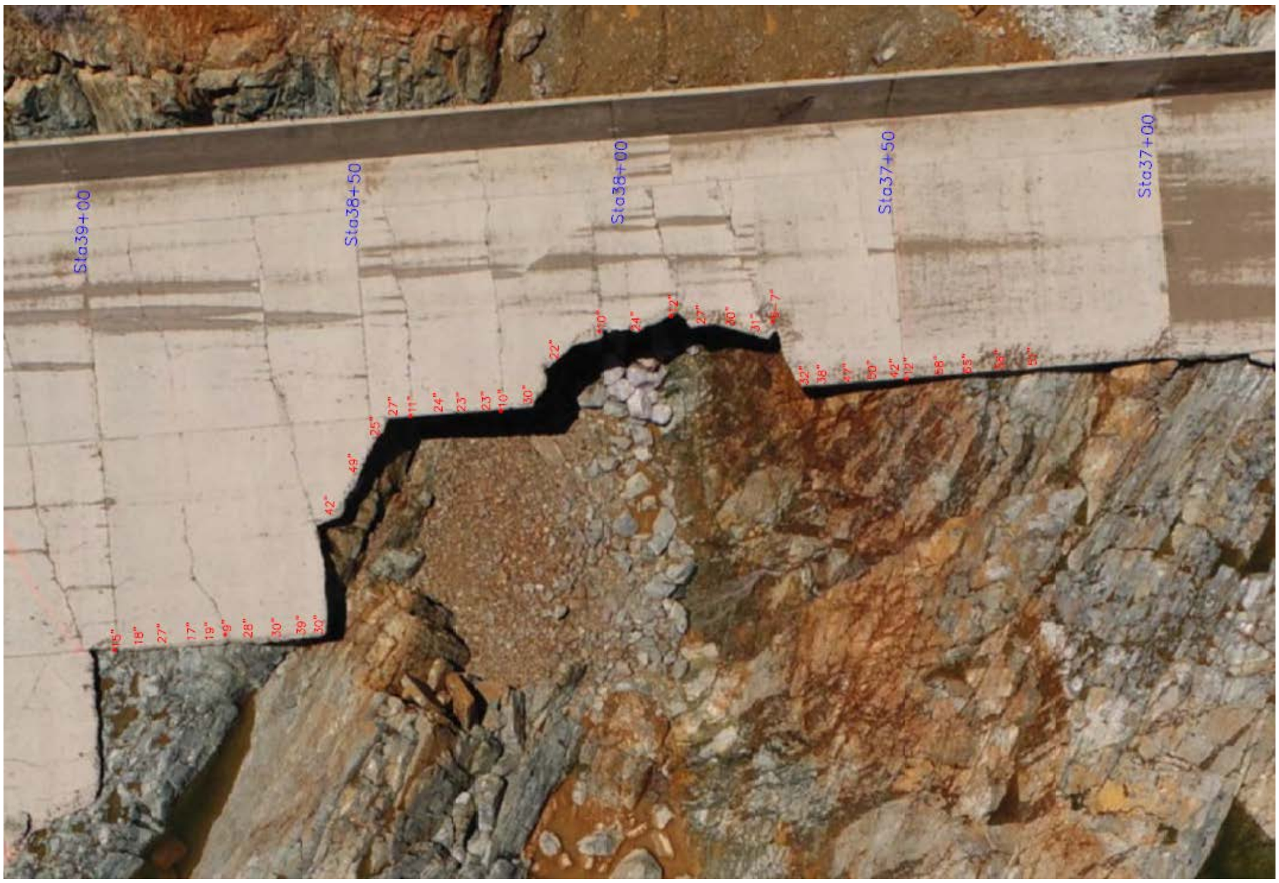



**Figure D-13: Plan of Erosional Feature Between Sta. 37+00 and Sta. 39+00 Showing Observed Concrete Thickness Values (in inches)**

The thickness of concrete remaining on each side of the erosional feature was observed to be significantly greater than it was in the central part. On the upstream part, from Sta. 37+25 to Sta. 37+60, the concrete was from 48 inches (4.0 ft.) to 65 inches (5.4 ft.) thick. In the central area, between Sta. 37+60 and Sta. 38+50, the concrete thickness values ranged from 22 inches (1.8 ft.) to 38 inches (3.1 ft.). Downstream of Sta. 38+50, the thickness values increased up to 49 inches (4.1 ft.).

A systematic evaluation of concrete thickness was not made in areas downstream of Sta. 39+00 because remaining slabs in these areas had been severely eroded by spillway flows heavily charged with entrained debris. The extent to which concrete surfaces had been planed down or eroded away was not known.

Discussion of Lower Chute Concrete Thickness. Based on observations made during forensic investigations and during demolition, it is concluded that most of the lower chute area was covered by an invert slab of a thickness more than the design minimum. However, except possibly at the sides of the chute in proximity to the chute wall foundations, it appears that there was no discernible compensatory thickening of slab concrete made during construction over areas with unfavorable and poor foundation conditions. These include some of the least favorable foundations seen anywhere along the spillway chute. Thus, although significant variations in concrete thickness have been observed in the lower chute, it is not possible to directly correlate concrete thickness with foundation quality or conditions at the time of construction. This can be considered in contrast to the upper chute area, where it does appear that, in some areas at least, efforts were made to adjust excavation depths (and concrete thickness) in response to foundation conditions encountered during construction.

## 3.8 Foundation Interface and Voids Beneath Concrete Slab

Another line of enquiry for the IFT was to evaluate whether voids might have developed under the slabs of the spillway chute due to the erosion of underlying foundation materials. It is known that materials were present under the slabs that would have been readily susceptible to erosion by flowing water through natural joints in the foundation, cracks, and/or interconnected gaps between the chute slab concrete and the foundation. Such erodible materials may have been either sandwiched between the bottom of the concrete slab and underlying stable foundation (e.g. the "compacted clayey fines" and materials identified at the base of the concrete core holes [D3], or could have been located *in situ* within the foundation itself. Both conditions have been observed and noted. It was not possible to determine if such conditions existed at the location where the February 2017 incident was initiated, because material at this location was washed away during and after the chute failure. However, studies were conducted to explore this further at other locations on the service spillway.

After Ground Penetrating Radar (GPR) surveys had been conducted over the remaining undamaged areas of the spillway chute, DWR exploration focused on what had been interpreted from the high amplitude GPR survey as "potential voids," and which included features with air gaps within or beneath the spillway concrete. Core drilling was conducted at each of 12 GPR anomaly locations. Of the 12 features identified as anomalies and "potential voids," only one core location (Core-6) showed an actual, physical anomaly, consisting of a void with about a 1-inch wide aperture along a small portion of the interface between the concrete-foundation rock [D-3].

The Core-6 location was further investigated as part of the forensic study, because it was the only place discovered that clearly had some kind of void at the concrete-rock interface. A trench (Area A, about 2.9 feet by 3 feet and about 2 feet deep) was centered over Core-6 and excavated into the underlying foundation bedrock. The foundation interface in this trench was characterized by intact concrete overlying and in direct contact with slightly weathered to fresh amphibolite rock. No voids or defects were observed. The void in Core-6 documented during earlier investigations is attributed to mechanical breakage along existing joints or fractures during drilling, or possibly the void was filled when the core hole was backfilled with concrete [D-3].

The forensic investigations included study of areas of “compacted clayey fines” left in place during original spillway construction. It was evident that these were present and broadly distributed in many areas. Although the thickness of these deposits was observed to be generally only a few inches thick, pockets were found up to 1.5-ft-thick. The frequency of concrete being in direct contact with foundation bedrock was found to be well below 50% in the forensic trenches. In places, the construction geology as-built records [D-4] show 50% to 100% of the surface in these areas to have been covered with a layer of compacted clayey fines.

Such materials, in addition to strongly weathered or decomposed bedrock that is actually *in situ*, would be vulnerable to erosion by flowing water, especially in proximity to underdrains, open cracks, and/or control joints in the chute slabs. It is noted that gravel rock placed during construction of the underdrains was poorly contained and appears to have spilled over broad areas adjacent to the drains, as evident in construction photographs and during the forensic investigations. Presence of loosely compacted gravel on top of the “clayey fines” could have made the susceptibility to erosion worse.

If one takes into account factors described above and, although factual evidence of significantly large voids was not found during the forensic investigations, there are strong reasons to judge that voids could have conceivably developed at any time since spillway operations first began or voids could have been eroded very rapidly if large quantities of water under pressure penetrated through the chute slabs.

### 3.9 Chute Invert Concrete Condition

The original specifications for construction of the spillway chute called for coarse aggregate with gradation from ¾ inch to 6 inch. A clean natural gravel was specified, with crushed material accepted within specified amounts. The concrete aggregate observed in the chute slabs appears to be from a natural alluvial source with the majority of particles consisting of fresh amphibolite and gabbro, similar to the foundation bedrock. The 6-inch maximum size is larger than would be used in current practice for similar reinforced concrete structures where 3-inch maximum size or smaller would be more typical. From observations made of sections of chute slabs, at either saw-cut sections or exposed sections in damaged or demolished areas, it appears that the presence of large aggregate and thick placements might have presented issues or constraints during concrete placement.

Field observations and interpretations include:

- Large aggregate, combined with the 7-inch cover over drains and the reinforcement mat above the drains with 3-inch cover, probably resulted in issues at these locations during concrete placement and screeding at herringbone drains that required increased effort to consolidate the concrete, especially near the chute centerline. The effects of this were observed at several locations particularly in central areas of the chute, see Figure D-14. At some locations, it appeared that troweling, possibly excessive, was done in order to provide a smooth concrete placement in this difficult area above drains.

**Figure D-14: Example of area of congested large-size aggregate over herringbone drain and reinforcement. Location at Sta. 42+00 and centerline. Area of repair patch concrete shaded (note smaller aggregate).**

- As concrete was being placed, larger aggregate could have damaged the vitrified clay pipe (VCP) herringbone tile drains.
- Large size aggregate made it difficult to form and place concrete at joint keys between slabs with probable segregation.
- Evidence of segregation was observed at several locations where large aggregates seem to have accumulated toward the bottom of lifts and finer aggregates in the upper areas. The larger aggregate could have presented difficulties in concrete placement through the single layer of reinforcement in the upper part of the slabs.
- Cold joints and poor contacts between multiple concrete placements are evident in many of the thicker concrete sections, see Figure D-15.

**Figure D-15: Example of poorly prepared joint (red arrows) between two concrete placements in loose block lodged at dentates; joint was associated with open honeycombing, about 2 feet below top of slab; maximum slab thickness was ~6.2 feet. Note base of concrete bonded to relatively fresh rock, which became detached when the slab was lifted from foundation.**

<u>Condition and Deterioration of Chute Concrete over Time</u>. The following general observations are noted:

- No obvious signs of cavitation were observed on the invert chute slabs still in place during IFT field investigations. Extensive damage had occurred to remaining slabs in the lower chute area, which was largely due to erosion from materials carried in the spillway flow, and due to deformation of slabs adjacent to areas where concrete had been lifted and torn out.
- No visual evidence of alkali aggregate reactivity (AAR) or silica alkali reactivity (SAR) was seen.
- Slight indications of soft water aggressivity were observed, but only on chute walls beneath drain outfalls. This would have involved localized softening and slight erosion of concrete matrix and exposure of aggregate on the chute walls. No evidence was seen of enlargement of crcks or joints in the chute slabs by similar processes.
- The most significant and widespread deterioration of the condition of the chute concrete has been extensive delamination. During forensic investigation of the chute slab concrete, evidence of delamination was observed at the majority of contraction joints and the majority of cracks associated with herringbone drains. This phenomenon is discussed further in Appendix G: Repairs.

- Aside from the cracking over herringbone drains and spalling near joints and cracks, cracking of the chute concrete was relatively minimal.

## 3.10 Chute Slab Concrete Reinforcement

A single layer of #5 steel reinforcement bars was placed 12-inches on center each way in the upper part of the chute slabs. No corrosion protection was provided, since the project was designed and built prior to when this became common practice.

Condition and Deterioration of Concrete Reinforcement over Time. Reinforcement corrosion had been reported at some locations in previous inspection and repair documentation, as discussed in Appendix G. During the forensic investigations several examples of corrosion and failed bars were witnessed in cracks overlying herringbone drains, Figure D-16. In places, the bars appeared to have failed in tension from temperature loading (contraction). Corrosion at the cracks probably weakened the bars, making it easier for them to fail. The frequency of herringbone drains (approximately every 20 or 25 feet along the chute) means that what little benefit the single layer of reinforcement provided was seriously compromised. Severe corrosion was only observed at locations where flowing oxygenated water had come into direct contact with the steel, such as at cracks in the concrete, and not within intact concrete as has been indicated in some media reports.

**Figure D-16: Example of failed reinforcement steel with corrosion at a crack above a herringbone drain. The reinforcement is completely severed.**

Dowels at contraction joints were also observed to be quite severely corroded in places. This was generally at locations where the dowel sleeves had become damaged or where the dowels themselves had become damaged or exposed during concrete repairs. Corrosion was likely more severe because these dowels were not bonded to the concrete, and therefore more vulnerable. The original grease packing in the dowel sleeves had disappeared a long time ago. During the forensic field investigations, evidence was also seen where concrete reinforcement at herringbone cracks and dowels at contraction joints had been saw-cut during earlier repair programs but had not been replaced or spliced, see Figure D-23.

## 3.11 Joints

Several types of joints were constructed in the chute slabs. It is believed that most of these were intended to be contraction joints. A detailed discussion of joints can be found in Appendix A.

Longitudinal contraction joints. These were parallel to the chute alignment and were constructed as shown in Figure D-17. The central longitudinal contraction joint separating the left and right sides of the chute, between Lanes 3 and 4, was constructed as shown on Figure D-18.

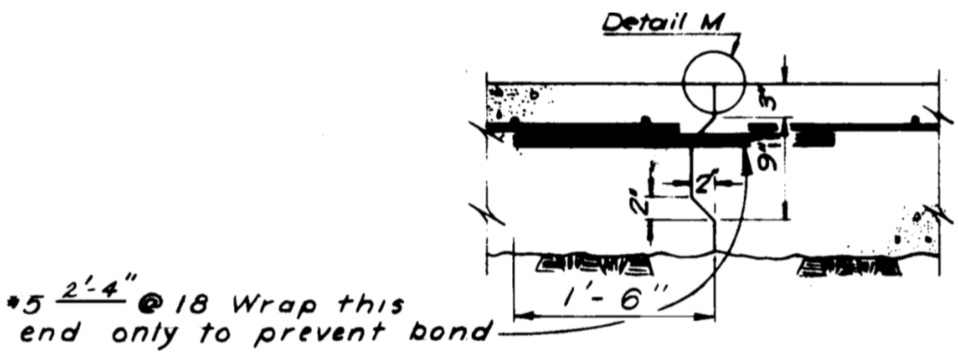



**Figure D-17: Longitudinal Contraction Joint [D-10]**

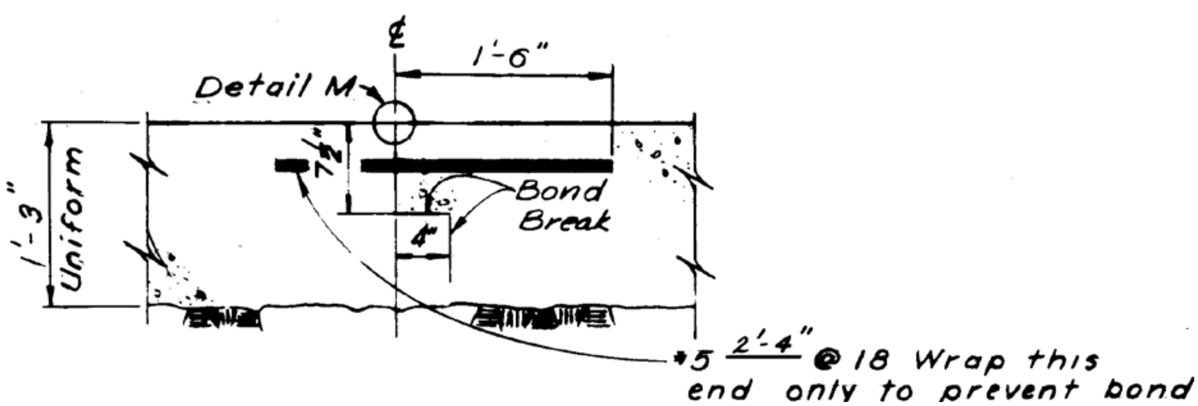



**Figure D-18: Longitudinal Contraction Joint at Centerline [D-10]**

All longitudinal contraction joints were configured with keys and dowels, although some variations in the geometry and dimensions of the keys were observed during forensic investigations and during demolition. Sheathed dowels were seen to have been installed consistently as shown on the drawings. These would have been packed with grease but this had long disappeared over the time since construction. No waterstops were installed in any of the longitudinal contraction joints, except on the spillway chute between the dentates. Drawings also indicate waterstops were installed at the only chute slab expansion joints, located between the headworks and chute slab[D10].

The overall condition of longitudinal joints observed during IFT investigations was generally good, with comparatively few concrete repairs (or patches) made to them, in contrast to the large number of repairs made to spalls at lateral contraction joints and to spalls at cracks over herringbone drains. Some cracking was observed partially crossing some of the rectilinear (centerline) keys but this was not as common or as extensive as that seen in lateral contraction joints.

Historically various types of elastomeric sealing materials had been placed into the joints at the chute invert, but these appeared to have had limited durability. No evidence was seen of grout injection into longitudinal joints, although non-shrink grout was reportedly used in some repairs, see Appendix G.

Lateral (Transverse) contraction joints. Lateral joints included:

- Formed lateral contraction joints positioned every 100 feet along the chute.
- Additional lateral contraction joints cut into new concrete at 50-ft intervals between the formed joints.
- Lateral contraction joints at 400-ft-spaced intervals (Stations 17+00, 21+00, 25+00, 29+00, 33+00, 37+00, 41+00, and 43+00). These joints were meant to have been installed with downstream offsets as shown in Figure D-19. The lateral contraction joints at 400-ft-spacing are coincident with expansion joints in the chute walls and had been called expansion joints at various times by DWR, see Appendix A.

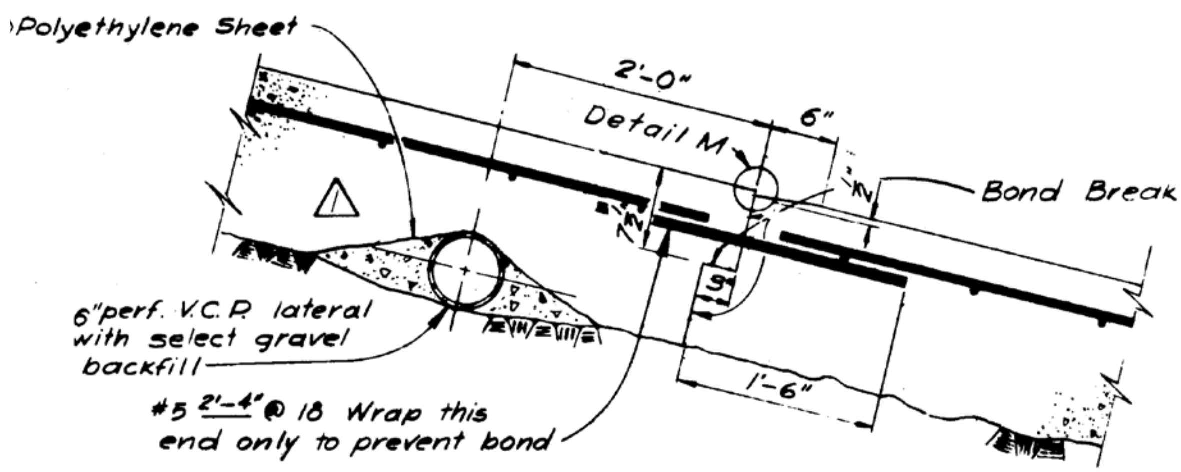



**Figure D-19: Formed Lateral Contraction Joint**

Formed offsets, as indicated on the drawings and Figure D-19, were not observed as consistent or uniform features at lateral contraction joints. The large majority of lateral joints observed during forensic investigations had no obvious evidence of offset. Offsets were observed at some lateral contraction joints at 400-ft-spacing, some of which possibly could have been installed during post-construction repair programs though there is no mention of such in repair documentation.

Most formed lateral contraction joints were constructed with keys and dowels, as shown on Figure D-17, although several variations in the geometry of the keys were observed during forensic investigations and demolition. Some lateral contraction joints showed no evidence that keys had been installed during construction, e.g. Sta. 27+00 and Sta. 39+00, Figures D-20 and D-21.


**Figure D-20: Lateral Contraction Joint at Sta. 27+00; Joint Key Absent**

**Figure D-21: Lateral Contraction Joint at Sta. 39+00 on Left Side of Photograph with Missing Joint Key**

Continuous reinforcement passed through locations where lateral joints were cut into the top surface of concrete at 50-foot stationing between formed joints. Forensic investigations revealed that vertical cracks had developed at only some of these joints. Possibly this was because the concrete was thicker at these joints compared with thinner concrete above nearby herringbone drains where cracking would have occurred preferentially.

The as-built construction drawings indicate that all lateral contraction joints would be installed with a "joint sealer" to fill the top inch of the joint between the abutting slabs, see Detail M shown in Figure D-22 below. No evidence of the original sealing material was seen during the forensic investigations. Various types of elastomeric materials were used to seal the joints in subsequent maintenance and repair programs but most of this material was found to be no longer present.

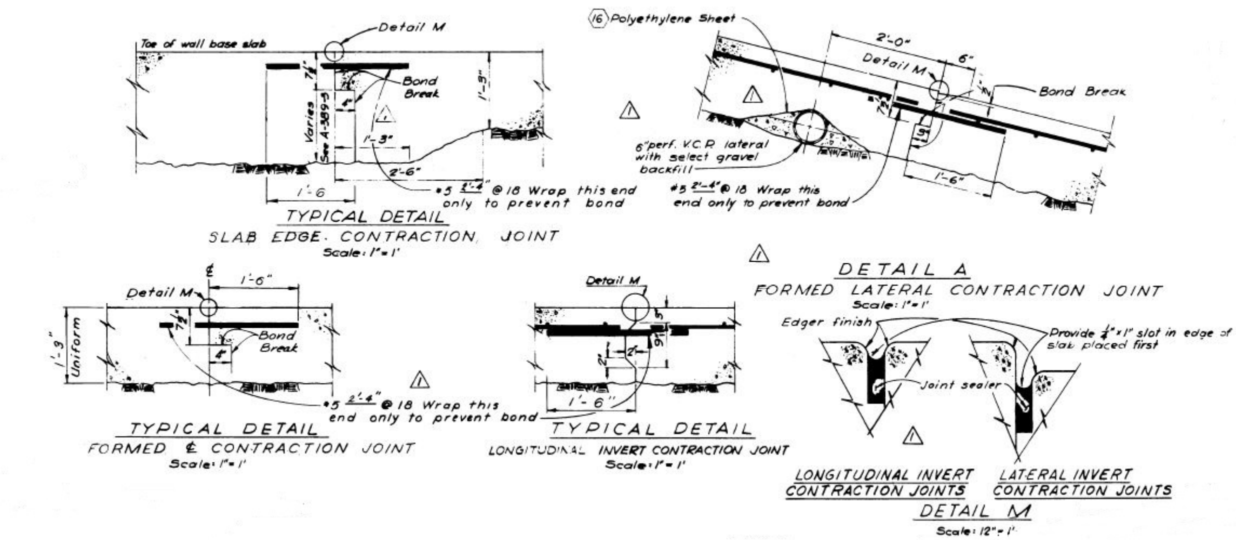



**Figure D-22: Contraction Joint Details [D-4]**

Reports have been noted of injection of grout into some joints during historical repair programs on the service spillway chute, as discussed in Appendix G. However, no evidence of cementitious grout materials was seen in any contraction joints during the forensic investigations, except in one possible instance at Sta. 43+00, where probable grout filling was noted in this lateral joint on the left side of the spillway. This is shown on Figure D1-23, which is an area of the lower chute that was highly eroded by spillway flows following February 2017 incident. Grout infill can be seen from an earlier repair program. In addition, dowels across the joint can be seen to have been saw-cut but not replaced or spliced during these repairs.

**Figure D-23: Lateral Formed Contraction Joint at Sta. 43+00**

As with the longitudinal joints, no waterstops were installed in any of the lateral contraction joints, except at the joints between the dentates. Waterstops were also installed at the only chute slab expansion joint (based on as-built drawings) at Sta. 12+97.24 at the downstream end of the headworks structure where it connects to the chute slab.

Condition and Deterioration of Contraction Joints over Time. Various types of degradation were observed associated with contraction joints:

- Stress failures at shear keys. During the forensic investigations, numerous observations were made of diagonal cracks cutting across the shear keys as illustrated in Figures D-24 and D1-23. This is a consistent and pervasive condition that could be properly seen only in the forensic trenches and at edges of concrete exposed by demolition or erosion. It would have been difficult (or even impossible) to detect such cracks during visual inspections at the surface of the chute. The condition appears to have mostly affected the right-angle (rectilinear) shear key configuration – as indicated on Figures D-24 and D-25. It is estimated that 80% to 90% of the joints with this configuration that were observed exhibited cracking of this nature to various degrees. Cracking was observed mostly above the horizontal section of the joint than below it, and in only one case cracking was seen both above and below the horizontal section. Cracking occurrence on the longitudinal joints appeared to be slightly less common than on the lateral joints. Although it is not known when the process would have begun, it is assumed that the condition developed progressively due to loadings exceeding the strengths of the keys. The result would have been reduced capacity of the shear keys and reduced ability to resist uplift forces applied to the slabs. In addition, it is possible that the hydraulic conductivity of some joints could have increased as a result of the cracking and shortened the seepage path between the top and bottom of the slab.

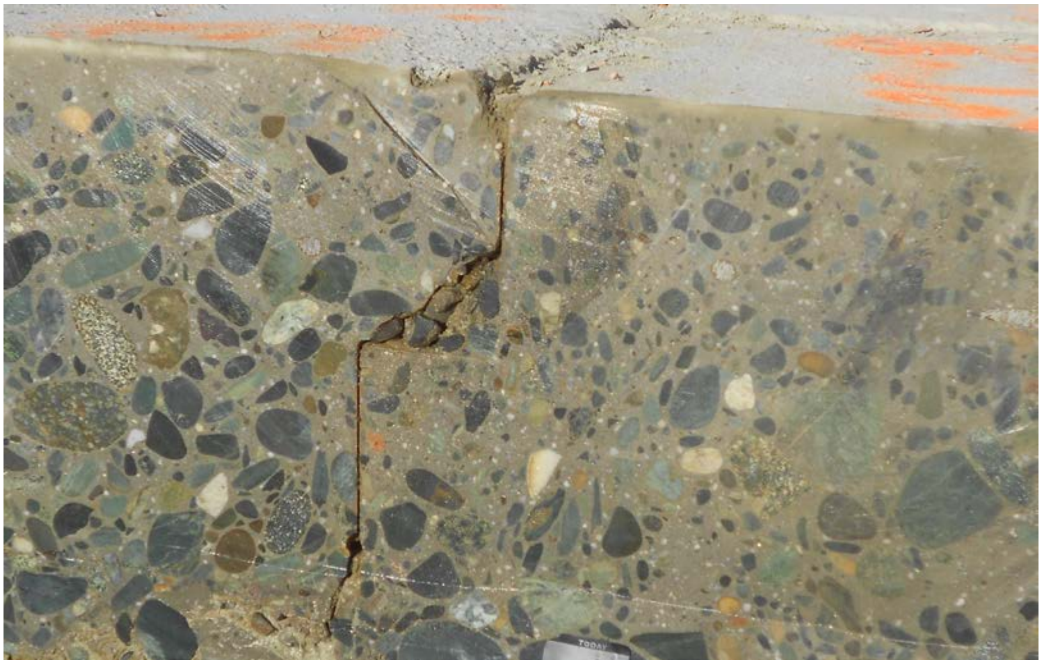


**Figure D-24: Typical Cracking at Shear Key**

- Bond at shear keys. According to the as-built drawings [D4], a bond breaker should have been applied on the concrete surfaces within the shear keys. It appears that, if the bond breaker had been applied, it was not effective in parts of many keys, in particular some parts of the horizontal and upper vertical portions of the key, as shown in Figures D-24 and D-25 (see yellow arrows). These portions appeared to have bonded (e.g. during low-flow/no-flow periods, by deposition of hydroxides and carbonates exuding from the concrete). Such cementation could also have contributed to the cracking described in the previous paragraph.

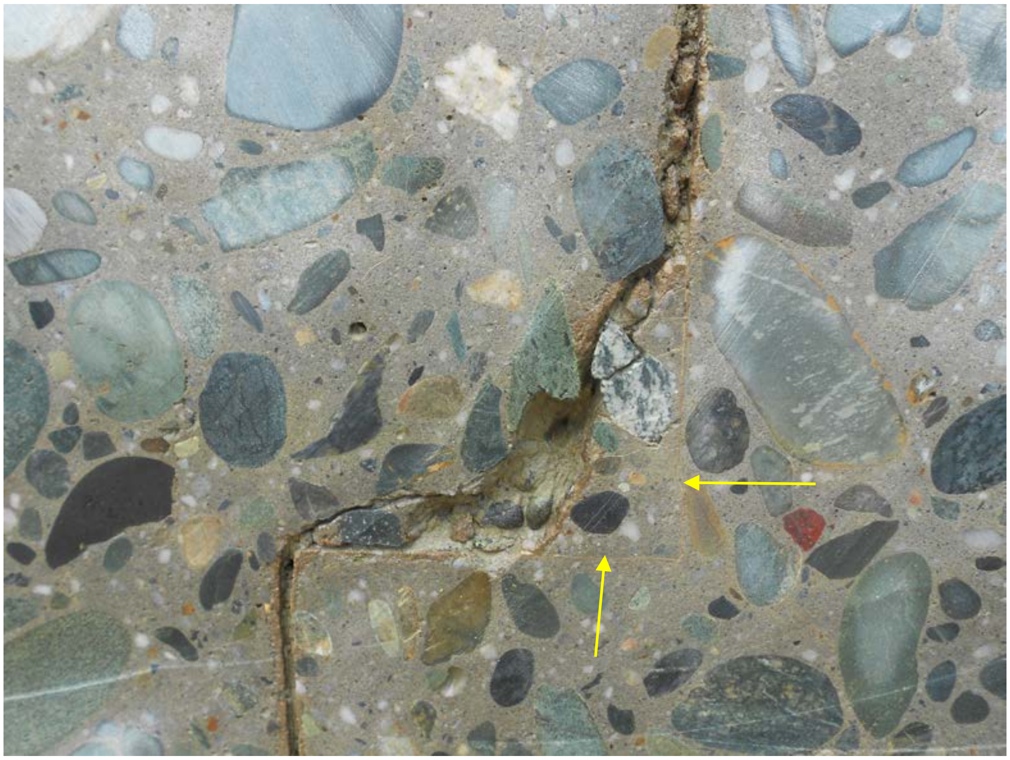


**Figure D-25: Shear Key with Tight Bond (Arrows) and Typical Cracking**

- Delamination. A common feature observed during the forensic investigations was the presence of delamination of the chute concrete. This feature consists of development of a horizontal crack ranging in depth from 3 to 6 inches, just above or at the steel reinforcement and dowel line. Typically, it was observed to extend away from a contraction joint or a crack located above a herringbone drain for a distance ranging from a few inches up to 2 feet. Figure D-26 shows an example of delamination extending from a crack over a herringbone drain encountered in forensic trench 26F1 at Sta. 26+37. Figure D-27 illustrates an example of delamination extending from a centerline contraction joint at Sta. 27+11. To the left of the contraction joint, a delamination is present at the base of a patch (at a depth of about 8 inches) and another delamination is running through the patch at the reinforcing steel level (at a depth of about 3 inches). Another delamination is starting to the right of the centerline joint extending horizontally about 6 inches at a depth of about 3 inches, just above reinforcing steel.

  Based on the forensic investigation and reports from historic repairs, delamination cracks were seen to have developed mostly on the downstream side of lateral contraction joints or herringbone cracks, though some instances were also observed on the upstream side (estimated approximately 75% on downstream side and 25% on upstream side). Evidence of delamination was also observed extending laterally away from longitudinal contraction joints, in particular the centerline joint which has a similar configuration as the typical formed lateral joints.

After a period of time, it is presumed the concrete would start to spall from delaminated areas and the chute invert would require repairing with patches. From discussions with DWR staff who had conducted chute inspections (in 2009 and 2013), this condition had been previously detected and most of the chute slab "drumminess" reported by DWR inspection staff was probably attributed to delamination at various stages of development.

**Figure D-26: Delamination Extending from Crack over Herringbone Drain Encountered in Forensic Trench 26F1 at Sta. 26+37**


**Figure D-27: Delamination Extending from Centerline Contraction Joint at Sta. 27+11**

- Failure of Repairs. An apparent recurrent theme with the contraction joints and the cracking over the herringbone drains is the fact that cycles of repairs were undertaken repeatedly without significant lasting effects, see Appendix G for more discussion. Evidence of this theme could be readily witnessed during the forensic investigations. Typical repeated repair efforts included:
  - Attempts to seal joints with a sealing compound or by grouting in order to reduce the amount of water entering joints. These were thought to be effective initially, but evidently were not and did not compensate for the lack of waterstops in contraction joints; and
  - Several campaigns to repair spalls and delamination by patching. These were also evidently not durable and did not address a basic condition causing delamination and spalling to occur and recur. Also, based on IFT interviews, it is likely that some of the repairs were not done in accordance with the specifications and/or material product instructions (e.g. improper use of bonding agents), see Appendix G.

  Although the poor durability of the repairs seems to have been recognized by DWR personnel based in interviews by the IFT, it does not seem that a concerted effort was made to determine basic underlying factors leading to conditions, such as delamination, and to develop more lasting solutions. This approach might suffice for a non-critical structure but for a spillway on a major dam project it is not appropriate. As can be seen in illustrations such as Figures D-24, D-25, and D-27 some joints have become increasingly compromised and vulnerable to spillway slab failure modes.

## 3.12 Foundation Anchors

Anchors (No.11 bar) were installed in the chute slab as shown on Figure D-28 [D-11] at 10-ft-spacing each way.

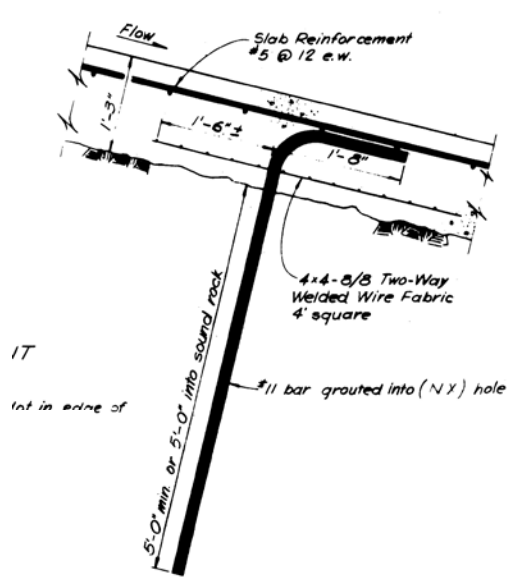



**Figure D-28: Typical Anchor Bar Installation**

At the time of design, no corrosion protection was adopted, presumably with the assumption that the bars would be adequately protected by encapsulation in concrete or grout.

In areas where the service spillway had been damaged during post-incident spillway discharge, the following observations were made:

- Where bedrock was relatively fresh and strong, and the chute slabs had been completely torn away, anchors bars were left embedded in rock either intact (indicating sufficient capacity in rock but less sufficient development in concrete), or had failed in tension or shear (indicating high bond strength but quite insufficient anchorage capacity to resist uplift forces experienced on February 7, 2017).
- Where bedrock was moderately to strongly weathered, either erosion had resulted in the complete removal of all concrete and much of the foundation, including anchors, or the foundation had been eroded from underneath the slabs and the anchors were left exposed – examples discussed below.

Results of forensic investigations and Ground Penetrating Radar (GPR) surveys by DWR [D-3] confirmed that spacing was generally in accordance with the as-built drawings [D-11]. However, no information was available, or evidence seen during forensic investigations to indicate that spacing had been adjusted or modified in response to changes in foundation conditions encountered during construction.

Similarly, from observations made during the forensic investigations, it appears that there was no systematic change in the length of anchor embedment in the foundation, such as should be expected in response to changes in rock condition. Almost universally, the anchor depths drilled into rock appear to have remained at 5 feet as shown on the as-built drawings [D-11]. This is illustrated in photographs taken in areas where the foundations were arguably the poorest quality of anywhere along the spillway chute, e.g. Sta. 31+50 to Sta. 34+50 and Sta. 37+00 to Sta. 38+00 – see Figures D-29 and D-30 respectively. Anchors can be seen in Figure D-29 with consistent and uniform spacing and lengths, even though the foundation in this area was geologically mapped during construction and classified as ranging from moderately weathered to strongly weathered and decomposed [D-4].

**Figure D-29: Undercut Area of Chute Slab at Sta. 33+00 (left side) with Exposure of Anchor Bars**

**Figure D-30: Undercut Area of Chute Slab at Approximate Sta. 37+00 (left) to Sta. 38+00 (right)**

Similarly in Figure D-30, which is a photograph of the area beneath scenes in Figures D-12 and D-13, anchors can also be seen with uniform spacing and lengths, even though the foundation in the area was mapped as moderately weathered to strongly weathered and decomposed and included a major shear [D-4]. In this photograph, the sloping surfaces that can be seen cutting into the slabs was produced by the plastic-wrapped gravel envelope surrounding the herringbone drains.

From observations made during the forensic investigations and demolition, it was apparent that bar hooks were L-shaped and placed beneath the rebar mat as shown on the as-built drawings. Construction records and photographs taken at the time indicate that longer, curved bar hooks were installed locally in areas of thicker concrete placement but none of these were seen during the forensic investigations. It is assumed that in areas where concrete was thicker than design (e.g. >3, 4, 5, or 6 ft. in thickness), the anchor bars had probably been increased in length to account for the thicker concrete. This is supported by observations made during demolition. However, no evidence was seen that anchors extended more than 5 feet below the bottom of the concrete, i.e. the 5-foot foundation embedment depth appears to have remained the same as shown on the drawings.

Observations made during the investigations and examination of photographs in failed (severely damaged) chute areas have provided information on the quality of anchor installation. Typically, good installation should have involved the following actions:

1. Drilling of a hole to the required depth and angle of inclination from vertical.
2. Making sure that the hole is stable and open, free of debris and loose materials and free of water (if possible). Hole stability and problems with loose materials can typically occur in poorer ground, intensely weathered rock, or if there is a layer of soil at the top of the hole.
3. Filling the hole with grout of the correct specified characteristics. If there is water in the hole, to make sure it is properly displaced by the grout.
4. Installation of the anchor making sure it is centralized and completely encapsulated in grout.

It is evident that at many installation locations these actions were not properly followed. As a result, anchor bars could be seen that were not properly encapsulated in grout, probably as a result of not properly fulfilling points 2, 3, or 4 listed above, or any combination of them. In the area shown in Figure D-29 (Sta. 33+00), photographic and video records show that most of the visible anchors were apparently well-encased in grout, though there are some possible indications of corrosion near the foundation-rock interface. In the area illustrated in Figure D-30 (approximately Sta. 37+00 to Sta. 38+00), consistent grout encapsulation was not as evident from photographs, drone imagery, and field observations. The anchors in the closest row seem to be reasonably well-encapsulated and, even where the grout has been chipped/knocked off (during erosion following the spillway chute failure), the underlying steel looks fresh and apparently uncorroded (e.g. anchor on extreme left of image and anchor second from right). In contrast, the next row of anchors behind are in poor condition with poor encapsulation, and exhibit moderate to severe corrosion.

During demolition, other observations in this area revealed even more severe corrosion of two anchors, suggesting that perhaps the anchors might have been exposed to aggressive groundwater for some time, possibly in one or more voids eroded in the foundation. In these places, it is possible that corrosion had been taking place for some time before the February 2017 incident. The propensity for anchor corrosion close to and just below the base of the concrete is not uncommon if grout encapsulation has not been done correctly and is a condition that has been observed on other projects.

Improper or insufficient grout encapsulation can directly result in insufficient development of bond capacity and significant increase in potential for corrosion. This can result in failure to reach adequate load capacity. Given the age of the Oroville project, and the different standards at the time of design and construction, more attention should have been paid during periodic inspections and reviews to the condition of anchors, their durability, and potential loss of anchor capacity. Another factor at Oroville is the soil-like conditions of some of the foundation. If the foundation is clean, hard rock, any gap at the top of an anchor hole that is not fully grouted may be filled by cement paste as the concrete is consolidated. However, if soil-like material covers portions of the foundation, it can also cover the top of the grout column, creating a gap in continuity between the grout and cement paste.

## 4.0 SERVICE SPILLWAY CHUTE DRAINAGE SYSTEM

This section of Appendix D provides documentation of observations and information related to the service spillway drainage system.

The spillway drainage system is comprised of two principal components [D-12]:

- An underdrain system to collect seepage and control uplift pressures within the foundation of the spillway headworks and gate structure. Some of this arrangement is connected to the upper part of the spillway chute underdrain system. These components of the drainage system were not included in the scope of forensic investigations.
- An underdrain system to collect seepage and control uplift pressures beneath the service spillway chute slab and in the backfill adjacent to the chute walls.

The chute slab underdrain system consists of 6-inch diameter perforated vitrified clay pipe (VCP) lateral drains, arranged in a “herringbone” pattern (in plan), spaced at 20 or 25 feet (depending on location) along the chute, and embedded in gravel at the foundation surface, directly under the chute slab. These drains pass under the chute wall footings and discharge to 12-inch diameter perforated VCP collector pipes, located outside the chute at the base of the wall. The lateral drains under the chute are generally referred to as the herringbone drains.

The collector pipes extend downstream and discharge back into the chute through outfalls that penetrate the walls at locations near the tops of the walls. Rather than having a single, common collector on each side of the chute, there are twelve separate collector pipes on each side of the chute, each draining a section of the chute and wall.

The underdrain system is capable of collecting water from different sources including:

- Groundwater recharged from reservoir seepage coming through the spillway chute foundation and/or the headworks foundation.
- Groundwater originating from natural recharge.
- Leakage through open joints and cracks in the chute floor slab and side walls.
- Water infiltration into the wall backfill from surface runoff, rain, or seepage.

### 4.1 <u>Investigations of Drainage Systems</u>

Since the February 7, 2017, incident, investigations were made of the remaining undamaged elements of the spillway drainage system. These included:

<u>Ground Penetrating Radar (GPR)</u>. GPR investigations were completed between Stations 14+00 and 28+00 in the upper chute area. The first geophysical survey was completed by GEOVision Geophysical Services, between March 6 and 19, 2017. The survey consisted of ground penetrating radar and impulse echo methodologies. Results of the GPR survey were used to identify potential voids or anomalies within and beneath the spillway chute, steel reinforcing spacing in the concrete slab, approximate concrete thickness, and the surface traces of the VCP herringbone underdrain system. Subsequent GPR surveys were completed by Caltrans to obtain more details, including locating potentially shallow underdrains and drains that were potentially “fouled” (blocked in some manner). The results of the GPR surveys are described in a draft DWR Geology Report [D-3] provide to the IFT in October 2017.

<u>Crack Mapping</u>. From DWR reports, photographs, and satellite imagery prior to the February 2017 incident, it was evident to the IFT that nearly every herringbone drain under the service spillway chute was associated with a crack in the concrete immediately above a drain. Following sealing repairs in 2009, the visibility of the cracks was so conspicuous that they could even be seen on Google Earth satellite imagery [D-13]. The initial phase of the 2017 geophysical surveys included systematic mapping of crack traces on the concrete surface of the upper area of the spillway chute. This crack mapping was performed by DWR Project Geology using high-resolution, geo-referenced drone photography, Figure D-31. A similar survey of the damaged lower chute section was not performed, although high-resolution drone imagery was obtained and reviewed by the IFT. Results of the crack mapping are presented in the DWR Geology Report [D-3].

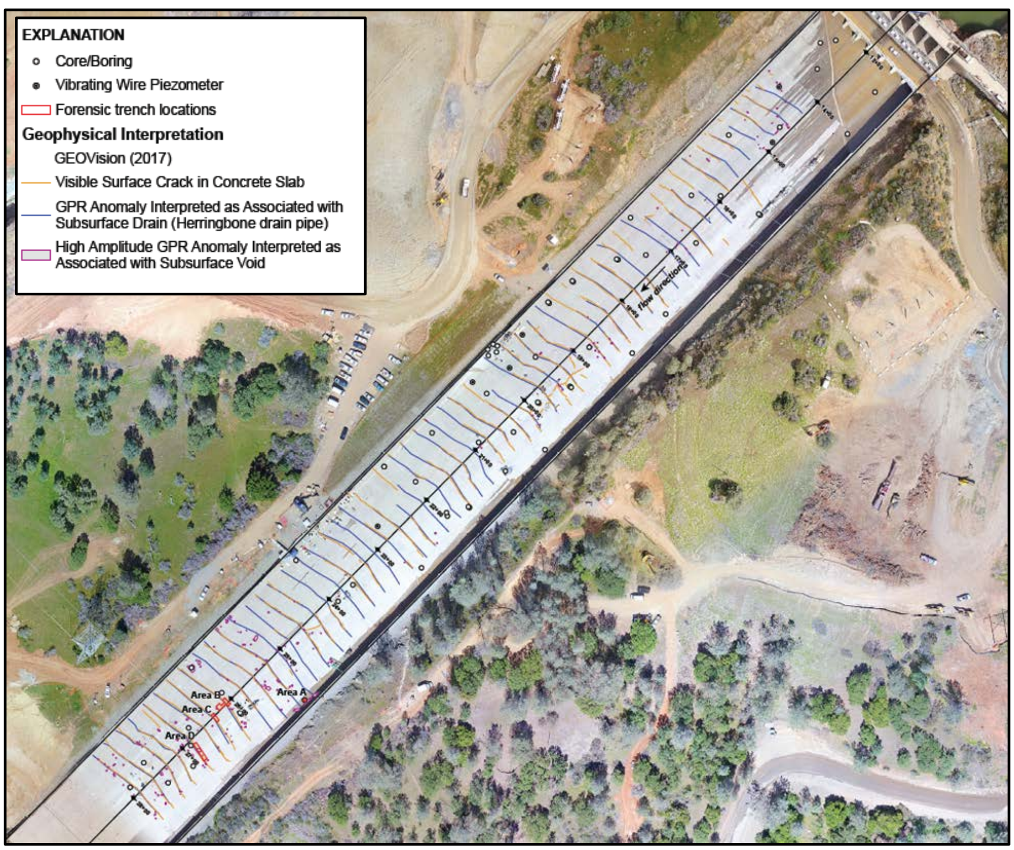
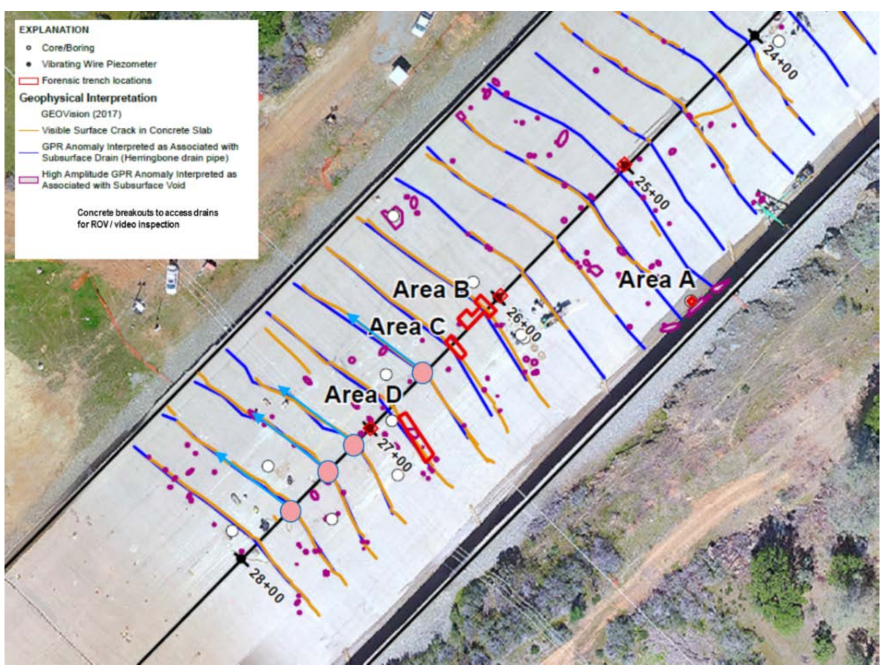



**Figure D-31: Crack Mapping in Upper Chute Area**

Video Surveys. HDR performed several remotely operated vehicle (ROV) / video inspections of the spillway underdrains and of the collector pipe system at various locations in the upper chute area, upstream of Sta. 28+00. These included inspections of the collector pipe system on both sides of the chute certain herringbone drain accessed through forensic trenches saw-cut into the chute slabs along the chute centerline, and a limited number of herringbone drains between Sta. 22+00 and Sta. 23+00 that had been identified by GPR survey as being potentially fouled. HDR also conducted a limited inspection of herringbone drains in the lower chute area where chute slabs were still in place. The only sections that could be inspected by ROV in this area were a small number of drains on the right side of the spillway between Sta. 40+00 and Sta. 41+00, which were accessed from outside the chute walls, where erosion had exposed the locations where the drains had intersected with the collector pipe system. Results of the HDR surveys are documented in DWR [D12] or were provided to the IFT in briefings from DWR and their consultants, June 20/21, 2017, and October 17, 2017 [D-14].

Forensic Trenches. Investigations of the herringbone drain system were carried out by DWR resources with IFT oversight. These included the removal of sections of concrete by saw-cutting to expose drains in an area of the upper chute between Sta. 26+00 and 28+00, see Figure D-30. Upon removal of sections of chute slab, the drains were carefully dismantled (deconstructed), mapped, and photographed. The foundations beneath were examined, the geologic conditions recorded, and samples of soil materials were collected for analysis. At four additional locations along the chute centerline (Sta. 26+57, 27+10, 27+31, and 27+58), the concrete was broken out (again by saw-cutting and raising slabs) in order to expose the underdrains. This allowed remotely operated vehicle (ROV) inspections of sections of drain. The locations of the forensic trenches and concrete break-outs along the chute centerline are shown on Figure D-32.




Note: direction of inspection and location indicated by symbol.

**Figure D-32: Location of Concrete Breakouts Along Chute Centerline in Upper Chute Area**

## 5.0 DRAIN CHARACTERISTICS AND DESIGN ASPECTS

The design characteristics of the drainage system are described and discussed in Appendix A of this report. Typical drawing details of underdrains are shown in Figure D-33 [D-11] and the

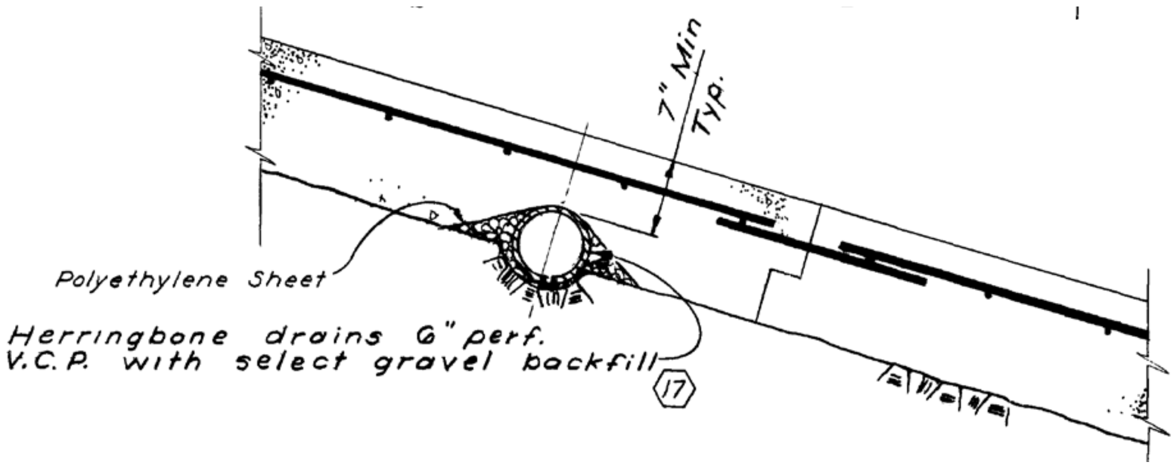



**Figure D-33: Typical Herringbone Drain Installation**

Photograph in Figure D-34 shows typical removal of a saw-cut slab from above a herringbone drain in the upper chute area at Sta. 26+12 and centerline. This illustrates the typical gravel pack at an underdrain, the polyethylene sheet (adhering to base of concrete), and a crack above the drain.


**Figure D-34: Removal of Saw-cut Slab from Above Herringbone Drain in Upper Chute Area**

The following paragraphs present observations and comments on design and construction aspects that were made during the course of the forensic investigations.

Drain Materials. Observed unfavorable aspects and factors include:

- VCP was specified and installed for both herringbone drains and the collector pipe systems. This was an acceptable standard product at the time but would not be considered suitable for the design purpose today for several reasons. VCP can be brittle and readily break if subjected to non-uniform loads or hard, sharp blows such as may occur during handling, installation, or even during operation. The HDR report on video inspection of the drainage system points out that the VCP pipes tended to have numerous structural defects (various types of fracture, cracking, holes) compared to other drain materials being used at the time (e.g. asbestos concrete pipe which was used in the spillway headworks). Cracking of the VCP was the most commonly identified structural deficiency of the pipe materials, followed by joint separations, misalignments, and mismatched or absent perforations.
- The bell and spigot connections associated with the VCP herringbone drains required a caulking compound to make the connections relatively watertight. For the herringbone drains a jute caulking, soaked in tar or grease was specified. In the forensic trenches this material was seen to have completely degraded over the service life, resulting in open, highly pervious connections. Although this effect might have improved the inflow capacity of the drains and their ability to capture water, it also resulted in increased potential to surcharge water pressures beneath the chute slabs and into the foundation. This could have increased the potential for erosion of decomposed rock and soil-like materials when, or if, the drains were flowing at full capacity.
- The collector pipe installations were specified to have mortared connections with some jute tamped into the connections prior to filling joints with mortar. At many of the connections observed in the video inspections reviewed by the IFT, it was seen that wet mortar had been liberally applied at connections, but it appeared that, when the connections were closed, excess mortar had squeezed into the pipe causing a minor hydraulic blockage. The mortar is also brittle upon curing, and any movement of the pipes resulted in cracking and, in many places, complete loss of joint infilling. During demolition of collector pipes, it was noticed that most mortar had been dislodged and jute materials were not found (it had either completely decomposed, washed out, or had never been originally installed).
- The specifications [D-10] stated "select gravel shall be placed around all perforated pipe…". The VCPs observed in the forensic investigation were invariably placed directly onto the foundation without being surrounded by gravel, including locations where the drains were placed directly on top of strongly weathered bedrock or "compacted clayey fines". This is not good practice by current standards and possibly not at the time of construction. Generally, where there are fines between the foundation rock and the gravel envelope there should have been a compatible filter material placed between the two materials to avoid migration of the fines into the drains. The select gravel backfill was specified to pass "a 1½ inch screen and be retained on a ½ inch screen." This gradation has

no filtering capability and currently would not be acceptable as a gravel pack around a drain.

- A polyethylene sheet was placed on top of the gravel to separate wet concrete from the gravel. In this respect, the measure was effective. However, the plastic sheet would have prevented water in cracks in concrete above drains from leaking directly to the underdrains.

Installation. The specifications called for the underdrains "to be laid directly upon the rock surface exposed by excavation …. In areas of over-excavation, pipe shall be brought to grade by the use of backfill concrete …." In such areas, as-built drawings and revisions during construction indicate that the hydraulic connection to the foundation was achieved by use of gravel-filled "sonotubes" or formed gravel-filled trenches, see Figure D-35.

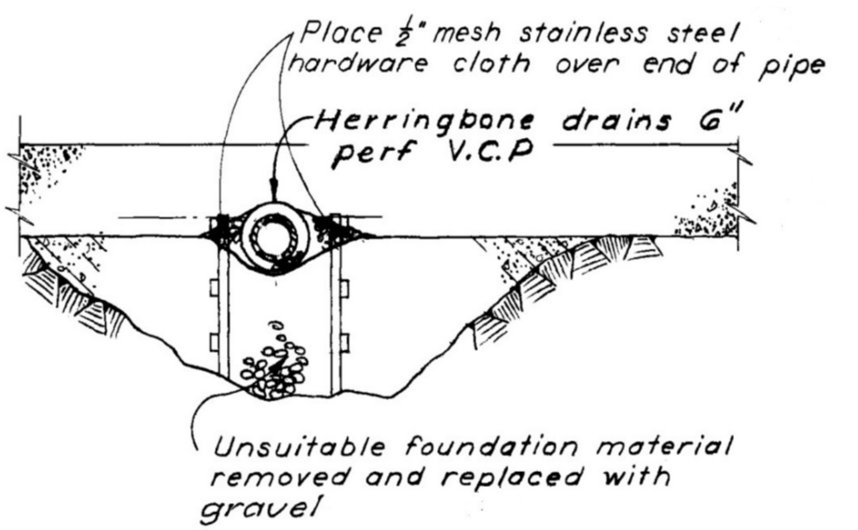



**Figure D-35: Concrete Backfill Drainage Detail**

Both the types of concrete backfill drainage detail were observed at numerous locations during the forensic investigations and demolition. It was noted by the IFT that these details were probably difficult to construct, and several deviations or defects were observed during forensic investigations, including:

- Collapsed gravel formwork or sonotube, resulting in a locally ineffective drain and/or uncontrolled hydraulic pathways through concrete or over the foundation. Figure D-36 illustrates drain construction defects at a longitudinal contraction joint, between lanes 2 and 3, at approx. Sta. 37+00 – Sta. 37+70 in the lower chute area. Underdrain locations are shown by yellow arrows, collapsed formwork and gravel spillage are shown by X symbols, and a suspect installation is marked by # symbol where gravel formwork was apparently 3-4 feet in width.

- Noted absence at several locations, over distances of 10 to 20 ft., of gravel formwork or sonotubes needed to connect drains to the foundation through thick concrete backfill.

- Frequent spillage of gravel from formwork or sonotubes.

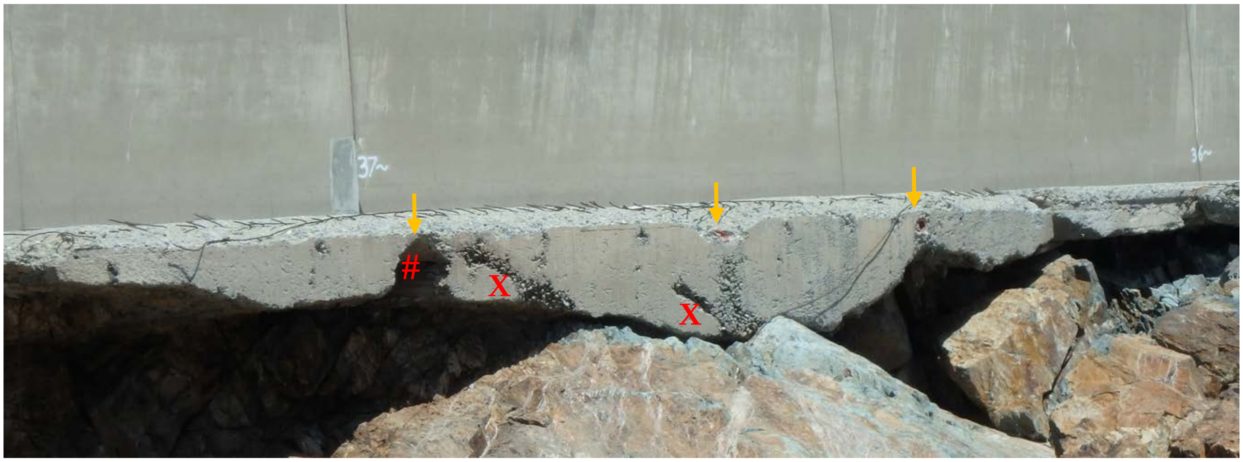


**Figure D-36: Typical Drain Construction Defects (explanation in text)**

## 6.0 SUMMARY OF RESULTS OF ROV/ VIDEO INSPECTIONS

The following is a summary of findings from ROV/ Video Inspections of the chute drainage systems performed in 2017. Naturally the conditions of herringbone and collector drains in the sections of spillway that originally failed (between Sta. 33+00 and Sta. 34+00) are not known and cannot be determined based on currently available evidence.

Cracks: Cracking of the VCP was the most commonly identified structural deficiency, as mentioned earlier in this appendix. The causes of cracking were not evaluated, and the cracks could be attributed to a variety of causes (e.g. non-uniform loads or hard, sharp blows such as may occur during handling, installation, or movements/settlement during operation).

Perforations: Mismatched perforations (pipes installed at incorrect orientations) or sections of pipe with no perforations were also observed at several locations. The IFT does not consider these to be major deficiencies because they occurred only locally over short distances and probably did not have a significant impact on drain performance.

Broken or Collapsed Pipe: None of the drainage system pipes that were inspected had collapsed, except in some locations where severe damage and collapse was attributed to nearby demolition activities involving blasting [D-14].

Open Joints: Open or defective joints were the second most common structural deficiency noted during the inspections.

Soil Infilling (Settled Deposits), Debris, Encrustation and Scale: The only area where significant deposits were found resulting in total blockage was in one herringbone drain in the upper chute area, see Figures D-37 and D-38 [both from reference D-14]. This was discovered during a follow-up video inspection to investigate potentially “fouled” underdrains that could have been filled with sediment and originally identified in a Caltrans GPR survey [D-14]. A 15-ft-long section of a herringbone drain at Sta. 22+50 was found to be completely filled with a lean clay deposit. This infill was sampled and analyzed but its origin has not yet been determined, though it could conceivably have originated from erosion of highly weathered foundation or from re-working of the known deposits of construction debris left on the foundation ("compacted clayey fines").

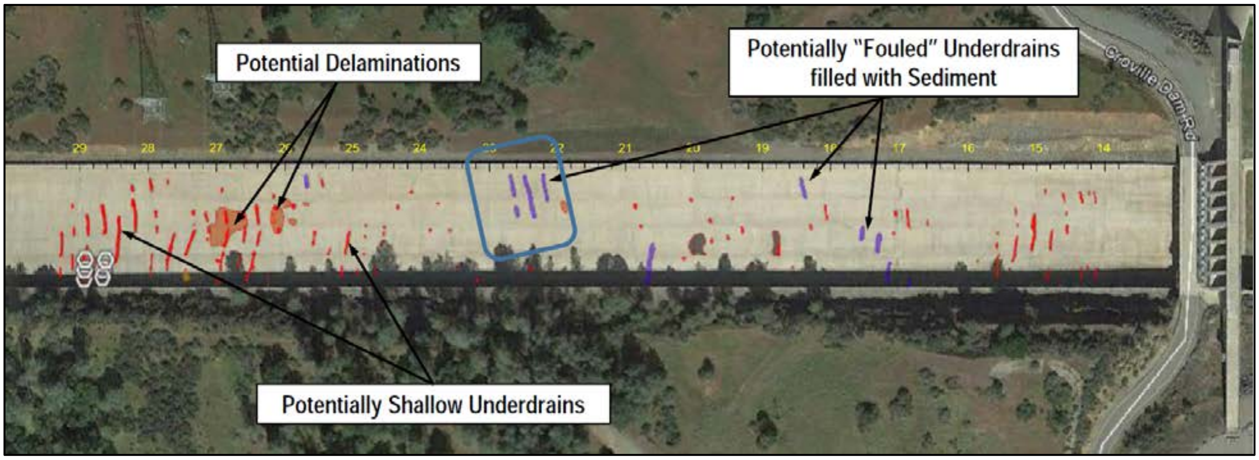



**Figure D-37: Location of Potentially Blocked Underdrains and Potentially Shallow Underdrains**

Elsewhere, only minor soil deposits were found, typically at separated joints. The small quantities involved would not have had any significant hydraulic effects in performance of the drains. Gravel and cobbles were intermittently present in the system. Gravel was found at offset joints and was assumed to have originated from surrounding gravel fill. Cobbles were present only below the collector pipe cleanouts and were presumed to have been dropped in. Other minor debris, soil deposits, encrustation, and scale were observed at several locations.

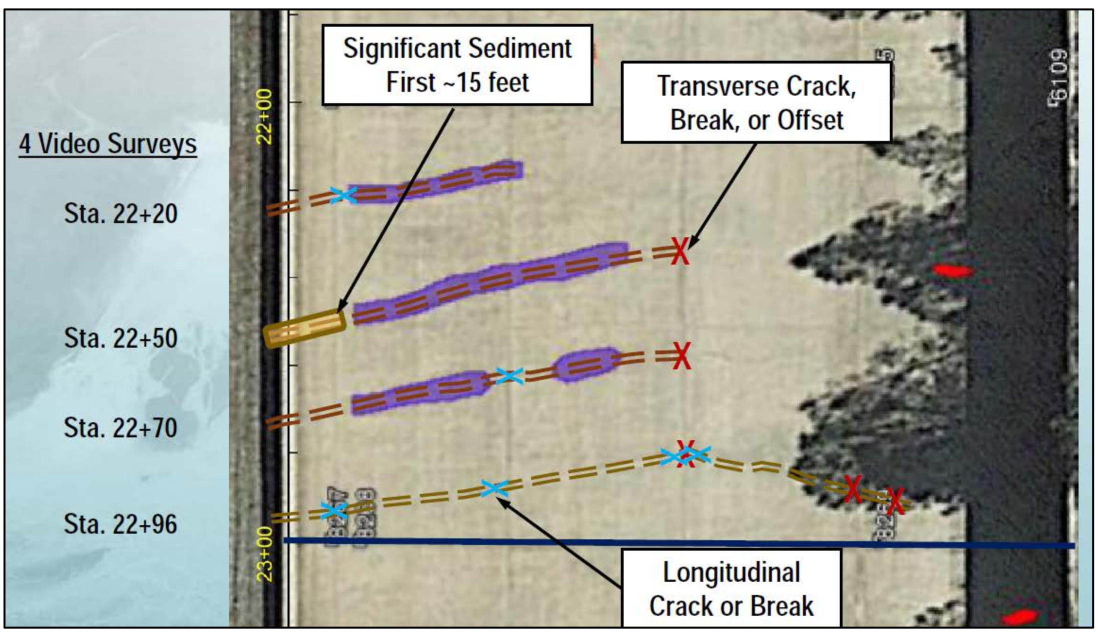



**Figure D-38: Follow-up Video Surveys of Herringbone Drains Sta. 22+00 to Sta. 23+00**

Seepage and Water: HDR reported that water was not present in any of the collector drainage systems on the right side of the spillway chute during the ROV inspection. However, collector pipes on the left side of the spillway were observed to have both standing water and flowing water. Standing water was held in place by minor soil infillings, encrustations, and minor line deviations. Flowing water entered from open joints and cracks and exited at other open joints and cracks further downstream.

Line Deviations: Deviations in the design alignment of the pipes were observed in several locations. These were considered relatively minor by the IFT and not significant to drain performance.

Roots: HDR noted minor tree root infiltration into the pipe at 15 locations. This topic was followed up by separate investigations on tree roots and reported by others outside the IFT.

## 7.0 FIELD TESTS AND OBSERVATIONS

Following the February 7th incident, HDR was requested to prepare a report summarizing the current understanding of the drainage system. In April, 2017, HDR published a report on their detailed study of the drains in the existing upper portion of the chute [D-16]. This included physical measurements of the collector drain outlets, the generation of a model of the drains using AutoCad, and observations of the flow depths at the collector drains.

The flows from the outfall pipes was observed for six outfalls on the left side of the chute and five on the right. The flow rates were determinded by observing the flow depths from a scale that was painted on the chute walls, Figure D-39. The scale on the side of the wall is excellent, but it was drawn relative to the outside diameter of the outfall conduit and not to the inside diameter. Therefore, the measurements do not represent the flow depth in the outfall pipe. The outfall of the collector drain are made from HDPE pipe and not VCP pipe as specified in the design drawings. The dimensions of the HDPE pipe were not called out in the report nor the date when the VCP pipe was replaced.

**Figure D-39: Collector Pipe Outfall**

The results of the tests are shown in Table D-1. The drain outfalls labeled L1 through L6 are located on the left side of the chute looking downstream and those labeled R1 through R5 are on the right side. Between February 27th and March 17th, the report describes the slab joints as being sealed. However, the gaps in the joints and the length of the joint that was sealed is not given. In addition, the report hints, but does not state if the cracks over the herringbone drains were also sealed. Some of the repairs may have failed during the test flow. However, the report does not indicate that the chute was inspected following the test program.

During the gate closure, the flow was diverted away from the right chute wall by sand bags. After a few hours, the flow from outfall drains on the right wall was observed to stop. This indicates that no groundwater flow outside the chute walls was entering into the right drainage system. Therefore, the flow out of the outfall drains on the right side of the chute on March 17th, after the repairs, must have been coming from undetected cracks over the herringbone drains or joints that were not sealed properly. The flow was never diverted from the left side of the chute during a gate closure, so it is not known if groundwater flow is entering the drains on the left side of the chute.

**Table D-1: Flow Tests**

| Drain Outfall | Percent Full Height 2/27/17 09:00 Hours 40,000 cfs | Percent Full Height 3/17/17 16:30 Hours 40,000 cfs | Percent Change (decrease in flow depth) |
|---|---|---|---|
| L1 | 20 | 10 | 50 |
| L2 | 30 | 20 | 33 |
| L3 | 40 | 15 | 62 |
| L4 | 40 | 20 | 50 |
| L5 | 20 | 10 | 50 |
| L6 | 20 | 20 | 0 |
| R1 | 50 | 10 | 80 |
| R2 | 50 | 10 | 80 |
| R3 | 75 | 40 | 47 |
| R4 | 75 | 40 | 47 |
| R5 | 50 | 15 | 70 |

The collection areas are not all the same size. The flow from the outfalls per unit area show that the outfalls on the right side of the chute pass about four times more flow than those on the left. This may be due to larger gaps in the joints on the right side of the chute. Unfortunately, these data were not taken during the joint repairs. The joints on the right are exposed to more sun than those on the left because of the shading by the trees on the left side of the chute. This may have accelerated the deterioration of the caulk that is used to seal the joints.

Turbid water is observed flowing from the first two drains on the left during rain storms. The turbid water is attributed to muddy runoff from the road that enters the cleanout riser or the collection pipes.

## 8.0 REFERENCES

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[D-11] As-Built Drawings; Oroville Dam Spillway; Specification No. 65-09. State Water Facilities, Oroville Division, Butte County, California, September, 1968

[D-12] Oroville Dam Spillway Underdrain Inspection Report conducted by HDR for DWR, dated May 6, 2017

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# Appendix E

# Review of Spillway Chute Design Practices

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# 1.0 BACKGROUND

This appendix was prepared to review the design practices for chute spillways around the time of the Oroville Dam service spillway design, which was completed in 1965. To get a reasonable representation of the spillway design details from that period, a twenty-year time-frame surrounding the 1965 completion of the Oroville Dam service spillway design was investigated. While a few of the 110 spillways used as a comparison to Oroville were designed before or after the 1955 to 1975 time-frame, most were well within and well distributed within this period. Additionally, national publications such as the US Department of the Interior, Bureau of Reclamation's (Reclamation's) *Design of Small Dams* from 1960 [E-1], and the USACE's EM-1110-2-2400 from November 1964 [E-2] were also reviewed.

While the USACE publication was probably not available to the Oroville spillway designers, they were available during the period of construction, when key drain and dowel details were being revised. It is also believed that the foundation excavation and cleanup in preparation for concrete placement were relaxed during the construction, as discussed in Appendix A. However, the DWR was not able to locate any documentation related to this change during construction, other than statements made in construction documentation that do not match the specifications paragraphs. Reclamation's "Design of Small Dams" was available to the designers. It is not known if any of these documents were used by DWR staff for the design or design changes to the Oroville Dam service spillway.

## 1.1 Key Spillway Design Features

While spillways being designed in the 1960s did not include all the features that might be included in modern spillways, there were eighteen key features that the IFT considered to be important in this study, as delineated later in this appendix. While comparing spillways with similar unit discharge and flow velocity would be ideal, this information was not readily available on enough spillways to make reasonable comparisons to Oroville.

## 1.2 Comparable Spillways

As stated above, 110 spillways designed by DWR and other organizations were reviewed for comparison of design details to the Oroville Dam service spillway chute design. This was done to get a sense of how the Oroville Dam spillway chute design compared to other spillway chute designs from that period, in terms of physical factors that may have contributed to the spillway chute failure. Eighteen possible spillway key features were evaluated. Since Oroville Dam is one of the tallest dams of its kind, it was difficult to find a suitable sample size for comparison. Therefore, spillway width, length, flow volume, and flow velocity were not considered for this comparison.

It should be noted that some of the spillways in the study experienced their own incidents in the past, but none as significant as the Oroville Dam spillway chute failure in 2017. During the twenty-year time-frame surrounding the Oroville Dam spillway designs, spillway design practice was evolving, especially in the 1970s, when waterstops placed in spillway chute joints was becoming common practice.

The IFT would like to acknowledge the following organizations for making spillway design information available for this study: DWR's DSOD, Reclamation, the USACE, BC Hydro, PacifiCorp, and Stantec (for providing information on international spillways designed by Harza). Table E-1 provides a summary of the findings from the review of the 110 spillways in the study. Note that the Oroville Dam spillway was not included in this table under the DSOD spillways. Its information is entered in its own column.

## 2.0 HOW OROVILLE SPILLWAY COMPARES TO OTHERS

All the spillways in the study had open channel chutes in at least a portion of the spillway, and some had tunnels that discharged into chutes. The spillways in the study also had various control and terminal structures, so for comparison purposes, only the design features for the open channel chute portion of each spillway were considered.

While specific information about individual spillways evaluated for this study is not being provided for security reasons, the Oroville Dam service spillway was larger and has higher discharge capacity and flow velocity than most of these spillways. Given this factor, the IFT considers that it would be reasonable to expect that the Oroville Dam service spillway would have been designed to include all of the state-of-the-art features included for other contemporaneous spillways. In other words, the IFT would expect that the Oroville Dam service spillway design would reflect the best practices of the time.

### 2.1 Key Features Evaluated in this study

The following is a list of key features that were considered in the study, and a discussion of the importance of each:

#### 2.1.1 Waterstops at Joints

Waterstops at joints help prevent the flow of water through joints in the concrete flow surface. A joint with offsets into the flow creates a stagnation point that can result in water flowing into the foundation through the joint. Once in the foundation, if not adequately drained, uplift pressures can develop. The practice of placing waterstops in chute slab joints was not common at the time of the Oroville spillway construction (only 38% of the spillways surveyed had waterstops), and the spillway chute at Oroville did not have waterstops at joints. This practice became more common in the 1970s. Instead of using waterstops at the joints, it was common practice to form a ½-inch offset, lower on the downstream side of transverse joints, to prevent a vertical offset into the flow that would allow water to flow into the joint. However, spalling at joints due to high temperature loads or freeze-thaw deterioration can create an offset that allows flow to enter the joint, if a waterstop is not present.

**Table E-1: Comparison of Spillway Features**

| Spillway Features[1] | DSOD[2] | USBR[3] | USACE[5] | BC Hydro | Pacifi-Corp | Harza[6] | Summary[7] | Oroville |
|---|---|---|---|---|---|---|---|---|
| No. of Spillways Evaluated | 30 | 48 | 17 | 4 | 5 | 6 | 110 | 1 |
| Design Years | 1952-1979 | 1950-1972 | 1951-1969 | 1957-1978 | 1951-1967 | 1956-1977 | 1950-1978 | 1965 |
| No. w/ Waterstops at Joints (%) | 14 (47%) | 15 (31%) | 1 (6%) | 2 (50%) | 0 (0%) | 6 (100%) | 38 (35%) | 0 |
| Design Slab Thickness (in) (common size) | 8-44 (12-18) | 12-30 (12-18) | 12-48 (12-18) | 12-24 (24) | 9-14 (12) | 30-48 (39) | 8-48 (12-18) | 15 |
| No. w/ Offset Joints (%) | 13 (43%) | 47 (98%) | N/A | 0 (0%) | 2 (40%) | 3 (50%) | 65 (70%) | 1 |
| No. w/ Keyed Joints (%) | 23 (77%) | 47 (98%) | 1 (6%) | 3 (75%) | 4 (80%) | 5 (83%) | 83 (75%) | 1 |
| No. w/ Cutoffs (%) | 20 (67%) | 46 (96%) | 1 (6%) | 0 (0%) | 2 (40%) | N/A | 69 (66%) | 0 |
| No. w/ 1 Layer of Rebar (%) | 13 (43%) | 2 (4%) | 12 (71%) | 2 (50%) | 5 (100%) | 3 (60%)[6] | 37 (34%) | 1 |
| No. w/ 2 Layer of Rebar (%) | 16 (53%) | 46 (96%) | 5 (29%) | 2 (50%) | 0 (0%) | 2 (40%)[6] | 75 (69%) | 0 |
| No. w/ Continuous Rebar at Joints (%) | 9 (30%) | 3 (6%) | 8 (47%) | 3 (75%) | 1 (20%) | 2 (33%) | 26 (24%) | 0 |
| No. w/ single Dowel at Joints (%) | 1 (3%) | 30 (63%) | 1 (6%) | 0 (0%) | 1 (20%) | 2 (33%) | 35 (32%) | 1 |
| No. w/ 2 Layers of Dowels at Joints | 0 (0%) | 3 (6%) | 0 (0%) | 0 (0%) | 0 (0%) | N/A | 3 (3%) | 0 |
| No. w/ Anchor Bars (%) | 15 (50%) | 43 (90%) | 13 (76%) | 4 (100%) | 3 (60%) | 5 (83%) | 82 (79%) | 1 |
| No. w/ Rock Foundation (%) | 25 (77%) | 41 (85%) | 14 (82%) | 4 (100%) | 3 (60%) | 5 (83%) | 92 (84%) | 1 |
| No. w/ Soil Foundation (%) | 5 (17%) | 7 (15%) | 3 (18%) | 0 (0%) | 2 (40%) | 1 (16%) | 18 (16%) | 0 |
| No. w/ No Drains (%) | 2 (7%) | 0 (0%) | 1 (6%) | 2 (50%) | 0 (0%) | 0 (0%) | 5 (5%) | 0 |
| No. w/ Drains Below Slab (%) | 27 (87%) | 48 (100%)[4] | 15 (88%) | 2 (50%) | 1 (20%) | 3 (50%) | 96 (87%) | 0 |
| No. w/ Drains within Slab (%) | 1 (3%) | 0 (0%) | 1 (6%) | 0 (0%) | 4 (80%) | 3 (50%) | 9 (8%) | 1 |

Notes:

[1]Not all data for features was provided for all of the spillways. If the feature was unknown the spillway was not counted for that feature.

[2]DSOD spillways do not include Oroville, since the reviewed spillways are intended to be compared to Oroville.

[3]USBR (Bureau of Reclamation).

[4]Thirty-one (65%) of the Reclamation spillway underdrain systems evaluated were determined to be filtered. The remaining 17 drainage systems were either not filtered, or it could not be determined.

[5]One of the USACE spillways (Fort Peck) was constructed in 1940, but was reviewed because it is similar in length and discharge to Oroville.

[6]Information related to the number of rebar layers was not available (N/A) for one of the Harza spillways.

[7] Where data are noted as N/A the spillways were not counted.

N/A – Not available.

### 2.1.2 Design Slab Thickness

The 1960 version of "Design of Small Dams" recommends a minimum thickness of 8 inches for smaller spillways with concrete placed directly on rock. Most of the spillways surveyed for this study had slab thicknesses that fell within the range of 12 to 18 inches. The 15-inch slab at Oroville is within this range. However, when anchor bars are used, there should be adequate concrete thickness to develop the required anchor bar strength within the concrete. Based on current ACI-318 Code requirements, a hooked No. 11 bar would require an embedment of 15.4 inches to develop 30,000 psi tensile strength in 3,000 psi concrete. Accounting for concrete cover, a 15-inch thick slab is probably under designed for development of No. 11 anchor bars. Many spillways with slab thicknesses in this range seem to use smaller anchor bars, such as No. 8 bars, at a closer spacing. Since the IFT did not have design calculations for any of the spillways evaluated, it is difficult to determine if any of the designers intended to develop the full anchor bar strength, or rather were providing oversized bars to compensate for potential corrosion and loss of bar strength.

### 2.1.3 Offset Joints

As discussed above for waterstops, joints were commonly offset on the downstream side to prevent a vertical offset into the flow. A detail and discussion of this offset is included in the 1960 version of "Design of Small Dams." The Oroville Dam service spillway, like many spillways of that period (65% of the spillways surveyed), had these offsets indicated in the design. However, as discussed in Appendix A, it appears that these offsets were not constructed for most of the joints, and may have only been applied to joints every 400 feet along the chute (see Appendix D).

### 2.1.4 Keyed Joints

Keys are used at joints to prevent differential movement between individual slab panels. Keyed joints were provided in 83% of the spillways surveyed for this study. Reclamation and others typically provided a key that is integrated with a cutoff design, as shown in Figure E-1 from the Best Practices document [E-3]. This "key" prevents upward movement of the downstream slab panel relative to the upstream panel. Keys at transverse joints for the Oroville Dam service spillway chute provide an offset at a depth of 7.5 inches that prevents the downstream slab from moving upward (Figure E2). This key is not as robust as the key in Figure E-1 because it is not the full depth of the slab. Differential movements could more easily cause spalling of the upstream slab panel above the key for the Oroville detail.

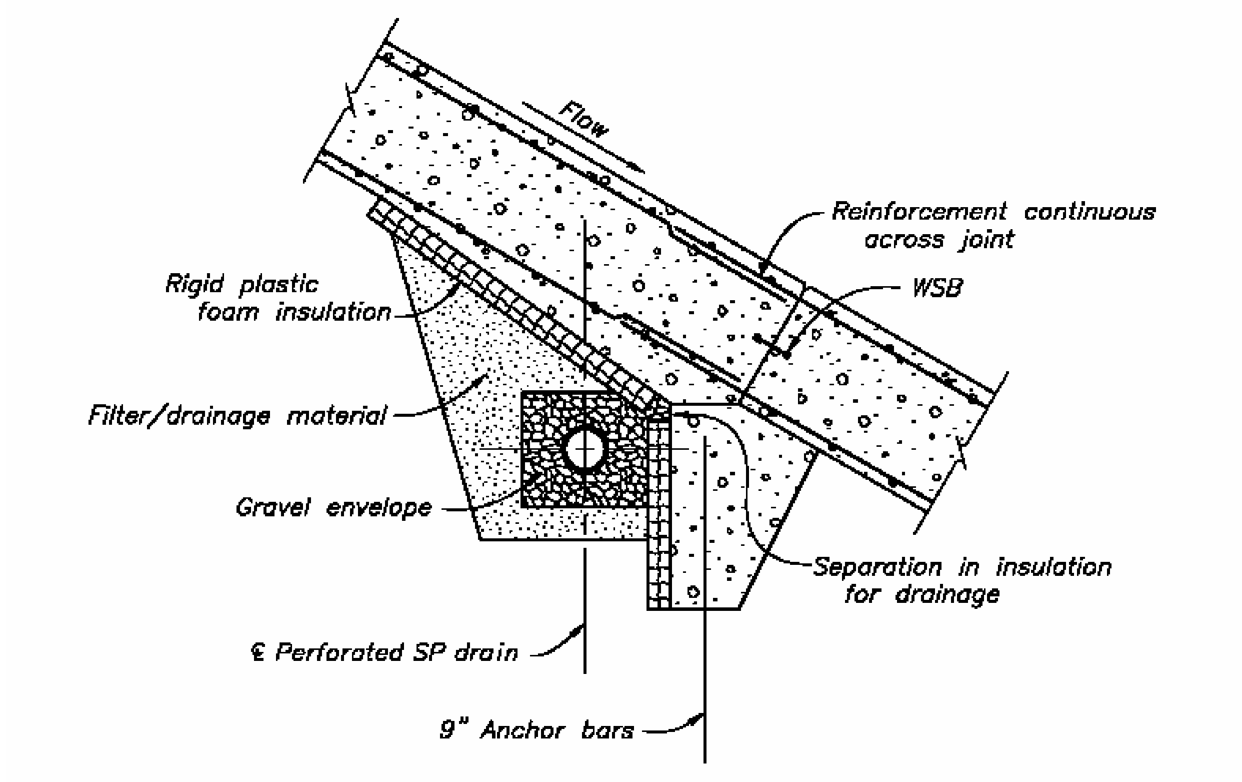


**Figure E-1: Typical Chute Joint Detail – Best Practices**

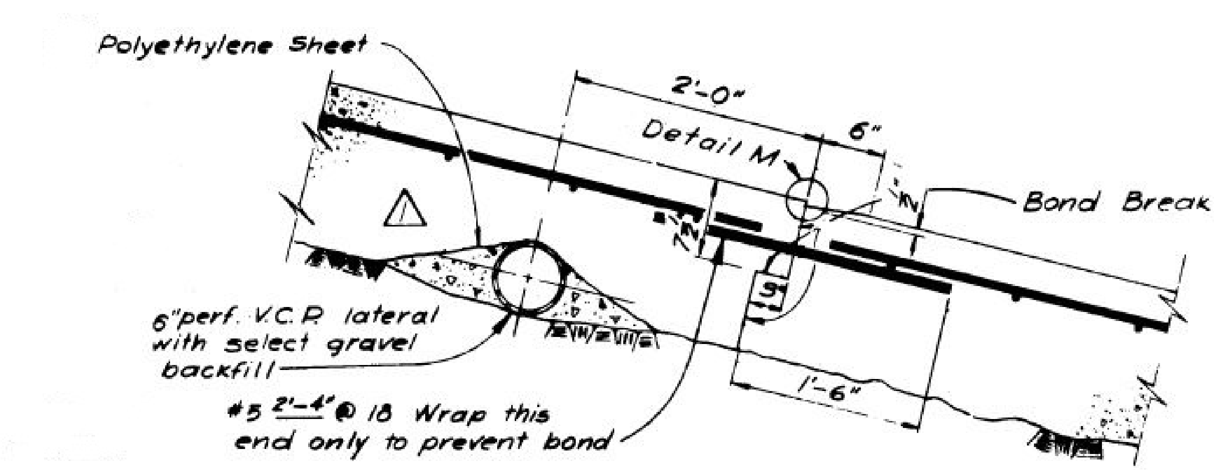


**Figure E-2: Oroville Dam Spillway – Transverse Joint Detail**

### 2.1.5 Cutoffs

Many spillways, even those with rock foundations, include intermediate cutoffs that extend below the slab and into the foundation (69% of the spillways surveyed). These cutoffs prevent movement of the chute slab on steep foundations, and can control the movement of water through the foundation. Figure E-1 shows a typical cutoff. The 1960 version of "Design of Small Dams" discusses using these cutoffs for soil foundations, and provides a figure with similar details to Figure E-1. All but two of the Reclamation spillways evaluated for this study had these cutoffs, which also act as a key, even when the foundation was in rock. Both of the Reclamation spillways without cutoffs were designed in the early 1950s, and one (Big Sandy) experienced a hydraulic jacking failure. Some of the excavated rock in the steep portion of the Oroville Dam service spillway chute likely had soil-like properties, and at one point the need for cutoffs (foundation keys) in the weathered foundation areas was discussed (see the Appendix A Timeline).

### 2.1.6 Rebar Layers

The spillways surveyed for this study had at least a top layer of rebar, except for one spillway in the DSOD inventory. The 1960 version of "Design of Small Dams" shows a single layer of rebar for a chute slab placed on rock, and discusses how slabs bonded to rock do not move much, but need temperature reinforcement on the surface. Apparently, this was the philosophy used for the Oroville Dam service spillway design. However, the Oroville service spillway chute slab foundation was not slightly weathered rock or better at all locations (as specified in the design). About 75% of the spillways surveyed for this study (including all but two of the Reclamation spillways) had two layers of reinforcement, even though 84% of the spillways overall were on rock foundations. For the spillways with two layers of reinforcement, the second layer was placed near the bottom of the chute slab.

### 2.1.7 Joint Reinforcement

The USACE's EM-1110-2-2400 suggests that slab panels that are keyed and anchored could be reinforced at the joints with slip dowels, like those included in the Oroville design. Without keys and anchor bars, it is better to have continuous reinforcement across the joints. Many of the spillways (32%) had a single layer of dowels at the joints, even if the slab had two layers of reinforcement. Fewer (24%) had continuous reinforcement at the joint, and still fewer (3%) had two layers of dowels.

### 2.1.8 Anchor Bars

Reclamation's *Design of Small Dams* suggested that stilling basins can be subjected to unbalanced uplift loads as the outside of the basin is saturated with tailwater, while sweep-out occurs on the inside. This same consideration could be given to spillway chutes for a gated spillway – if the foundation beneath the chute is saturated up to the depth of flow in the chute, and the gates are suddenly closed, there would be an unbalanced uplift. The USACE's EM-1110-2-2400 suggests that anchor bars be designed to resist a minimum of 5 feet of additional uplift pressure. It is believed that fully developed anchor bars at Oroville would have met this criterion. However, anchor bar embedments were not adjusted in the areas where the foundation was more weathered and fractured than in the anchor test areas. Seventy-nine percent of the spillways surveyed for this study had anchor bars.

### 2.1.9 Foundation Material

While the Oroville spillway was intended to be founded on clean, moderately weathered rock or better (according to the specifications), construction records suggest that this condition was not met for much of the steep chute section. While not common, some other spillways reviewed for this study had soil foundations and relatively steep slopes. Of the spillways surveyed with known foundation materials, 84% were on rock and 16% were on soil. The steep chutes designed on soil foundations generally were designed with foundation cutoffs that acted as shear keys against sliding.

### 2.1.10 Foundation Drainage

During the period when Oroville spillway was designed, the potential flowing water in the chute to produce uplift beneath the chute slab through hydraulic jacking was not well understood. The design of offset joints (as discussed above) was based on the concern that flowing water could enter the foundation, increasing uplift. However, the potential for high uplift and foundation inflows caused by stagnation pressures at joints and cracks was not quantified. Nevertheless, the Oroville Dam spillway chute drainage system was originally designed to remove ground water, and the entire piping system was increased in size after the contract was awarded to increase capacity. The survey of spillways for this study showed that 95% of the spillways had a drainage system for the foundation. However, only 9% had drains that were placed within the bottom of the slab, as was done at Oroville (Figure E-2). Of the spillways with drains within the chute slab, the drain pipes were a smaller percentage of the total slab thickness, than was the case for the Oroville Dam service spillway chute. For example, the one DSOD spillway with drains placed in the slab had a 6-inch half round pipe in a 21-inch thick slab (approximately 3 inches projecting into the slab, or 12.5% of the slab thickness) More than 50% of the 15-inch slab thickness was reduced at the drain locations for Oroville (see Figure E-3). This reduction in thickness lead to cracking over the drains. In most cases, when drains are placed within the slab there is enough room for a bottom layer of reinforcement to be placed above the drains. There was not enough room for this to be done at Oroville.

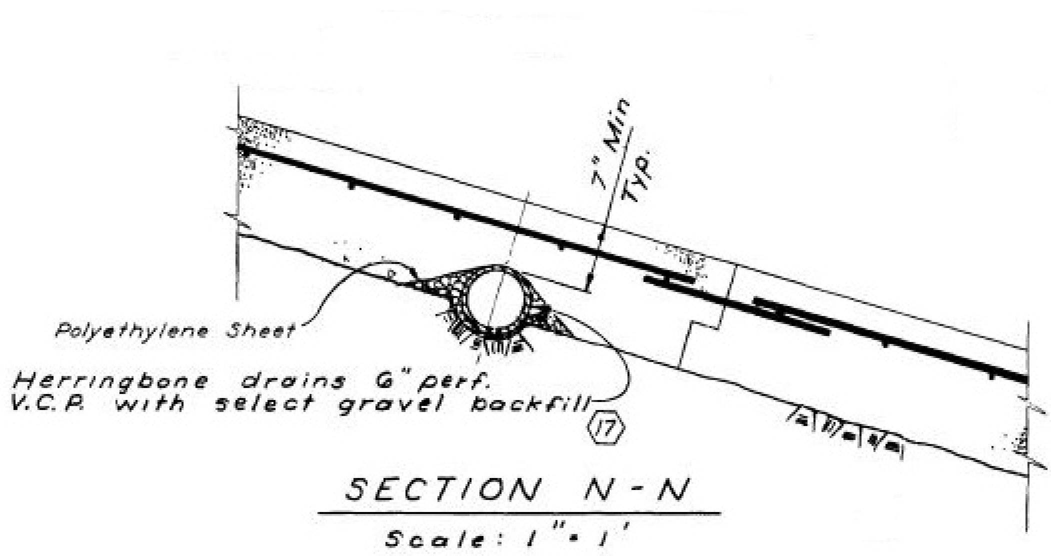


**Figure E-3: Oroville Dam Service Spillway Slab and Drain Detail**

Although information related to the longitudinal or collector drain systems was not formally collected for most of the other spillways considered, it was generally observed from drawings of the many Reclamation spillways that were evaluated had considerable redundancy in the longitudinal (collector) drains. While narrow spillway chutes (about 20 feet wide or less) typically had collector drains only on the outside of each chute wall, they were connected to both ends of the transverse drains, which were not oriented in a herringbone pattern. Wider chutes typically had one or more intermediate longitudinal drains beneath the chute slab, which also connected to the transverse drains. Some of the Reclamation spillways (11 out of 48) had drainage galleries constructed beneath the chute slab, typically near the chute centerline. These galleries were large enough for personnel access, and would not be easily plugged. Transverse drains enter the gallery from both sides of the chute. The Oroville spillway chute has a single collector outside each wall, which collects flow from transverse herringbone drains that extend beneath the side wall bases and for 80 feet under the chute slab, where they meet the drains for the opposite side of the chute at the chute centerline. Because the Reclamation drains are typically interconnected by outside and intermediate longitudinal drains and the drains are not in a herringbone pattern, if a drain were to be plugged at any location, there would be alternate paths for drainage around the plug. The herringbone drains at Oroville can only bypass a plug by flowing to the opposite side collector drain system. Since the herringbone drains are sloped to drain, this means that some drains would have to flow up hill. Also, there is no alternate path for collector drain flow, should the collector drains be plugged. In the case of the Oroville spillway failure, it is not clear if the lack of drainage system redundancy contributed to the failure. Even if the drains were not plugged, there may have been more drain inflow than could be handled by a typical, well designed drainage system.

## 2.2 Oroville Dam Service Spillway Chute Design Comparison

The purpose of this study was to see how the Oroville Dam service spillway chute design compares to other spillways from the same period, to see if the Oroville spillway performed as expected for a spillway of that time period based on known issues, or if there were significant design features were not included in the Oroville spillway design.

### 2.2.1 Features Similar to Other Spillways

The similarity of the Oroville Dam service spillway chute design to those of other spillways is based on the Oroville spillway having features that were typically included or not included in the design of spillways from that period. Some of the key features that were not often included in other spillway designs, or the design of the Oroville Dam service spillway chute, are waterstops at joints and continuous reinforcement at the joints. The Oroville spillway did not include waterstops at the chute joints, and it had slip dowels at the joints, both of which were typical for the period.

Some of the features that Oroville and other spillways from that period did have includes slab thicknesses of 12 to 15 inches (Oroville was 15 inches thick), offsets on the downstream side of transverse joints to prevent offsets into the flow (Oroville had these offsets at 400-foot stationing), keyed joints (Oroville had keyed joints, but some others had more robust keys, anchor bars, rock foundations, foundation drains), and dowels at joints (most spillways had either dowels or continuous reinforcement at joints). While the Oroville service spillway design and construction included dowels at transverse joints, some of these dowels were cut during the 1977 repairs.

### 2.2.2 Typical Features Not Included at Oroville

While 66 percent of the spillways surveyed from that period included intermediate cutoffs along the spillway chute, Oroville spillway had none. Other spillways having cutoffs included those on soil foundations as well as many constructed on rock foundations. These cutoffs, at the time, were considered necessary on steep chutes with soil foundations to limit movement of the chute slabs. The original Oroville design was intended to have a good rock foundation. However, as construction progressed in the steep section of the chute, it was apparent from the construction documentation (Appendix A) that the foundation had taken on more soil-like properties, and the possibility of adding keys was discussed by a DSOD inspector (see discussion in Appendix A).

The Interim Exploration Data report from 1964 [E-4] seems to discourage the excavation of keys in the foundation of Oroville spillway chute. Item 17 of the Tentative Conclusions and Recommendations states:

> "Excavation of the chute will produce a rough, irregular rock surface and shear keys are not thought to be necessary to prevent sliding of the chute lining. Blasting for shear key trenches in the foundation would only tend to weaken the rock by shattering."

It is believed that other similar construction projects would have had similar issues related to blasting for relatively small and shallow foundation shear keys. However, this issue was likely solved by contractors using jackhammers or other suitable excavation means. Also, as discussed in Appendix A, there were areas of chute foundation that did not seem to meet the description of the foundation above. These areas were weathered and the surface was mostly soil-like materials, where excavation of foundation shear keys may have been more easily accomplished without damage to the surrounding foundation.

It is unclear if adding cutoffs to the chute design would have prevented the Oroville chute failure. After the chute failed, opening of transverse joints in the chute above the failed section was observed, and anchors were added to this section to stabilize the slabs for the remainder of the 2017 wet season. Some movement on weaker, soil-like foundation surfaces likely occurred in the steep section of the spillway chute, which may have caused wider opening of joints and cracks in the area. Some of the chute sections downstream from the failed area were adequately tied to the rock foundation, limiting slab movement prior to failure.

The Oroville service spillway chute had a single layer of reinforcement, rather than two layers as was a more common practice at the time (69% of the other spillway chutes had two layers of reinforcement). As discussed, the design with a single layer of reinforcement may have been considered acceptable for slabs that are well bonded to strong rock foundations, but the as-built foundation at Oroville did not produce those foundation conditions. Therefore, it may be more appropriate to compare the details of the Oroville spillway in the steep chute section to spillways on poor rock or soil foundations. Because of the weak foundation contact bond strength, the slab could move more easily during temperature loading, resulting in larger crack and joint openings. An additional layer of reinforcement on the bottom of the slab could have helped reduce the openings at cracks. However, with the drains projecting into the slab, half or more of the slab thickness was reduced where the drains were placed, leaving no room for a second layer of reinforcement. While the IFT could find no specific references to placing additional reinforcement above drains that project into the chute slab, it would be expected that a structural engineer in the 1960s would understand that cracking is likely to occur where the cross section of a member has been significantly reduced, and that additional reinforcement may be required to maintain the member's integrity. Without any structural design calculations, reports from the BOC, or documentation related to design changes that occurred after the initial release of the specifications documentation, it is difficult to tell if impacts to the chute slab integrity resulting from changes to the drainage and foundation preparation and cleanup were ever considered by DWR.

### 2.2.3 Features at Oroville That Were Not Typical

Some of the features of the Oroville Dam service spillway chute design were not typical in detail to other spillways from that period. This includes a single layer of reinforcement and drains placed within the chute slab. While there were some spillways with similar features, these Oroville spillway details were the exception rather than the rule. While the details for a spillway on a rock foundation in "Design of Small Dams" from 1960 shows a single layer of reinforcement, most Reclamation spillways from that period had two layers of reinforcement. It is believed that the designers of the Oroville service spillway chute anticipated a foundation of good, strong rock and a chute slab that was well bonded (based on the specifications paragraphs and an interview with one of the designers), field conditions did not match this apparent design assumption.

The Oroville Dam service spillway chute design has herringbone drain pipes located within the slab and angled downstream, reducing the thickness of the slabs at the drain locations. While it is understandable that excavating trenches for drain pipes in a rock foundation may not be preferred, because it can be difficult and can result in considerable rock overbreak, the original bid specifications called for smaller 4-inch drains to be placed within the slab. It appears from the documentation (see Appendix A) that the drain pipe size was increased at some point after the original design and bidding, based on a recommendation by a Board of Consultants (BOC). It is believed that the lateral drains may have been placed within the chute slab for the same reasons that there were no foundation shear keys constructed for the spillway chute foundation as stated above. The 1964 Interim Exploration Data report seems to have discourages trench excavation in the foundation that would have been necessary to place the drains below the slab bottom. There also appears to have been no attempt to alter the slab design to accommodate the larger 6-inch drain pipe by either increasing the thickness and/or adding a second layer of reinforcement. While documentation provided to the IFT included references to the BOC recommendation, no reports from the BOC during the spillway design and construction were located by DWR, so it is unknown whether the BOC's recommendation to increase the drain size included other recommended changes to the slab and reinforcement design. While the decision to place drains within the chute slab, and even increasing the size of the drains within the slab, may not have been totally inconsistent with design practices at the time, this design feature was quite uncommon (only 8% of the spillway chutes considered had this feature). The decision to place the drains within the slabs and the apparent relaxation of foundation excavation and cleanup requirements were critical factors in the Oroville service spillway chute failure in February 2017.

Another of the features at Oroville that is not typical in a high-velocity spillway chute is the use of filler material to allow expansion at the invert joints. The use of this material is discouraged in the 1960 version of "Design of Small Dams," because it is difficult to maintain. The original bid specifications drawings had transverse expansion joints placed in the chute slab, and included ½-inch expansion filler material through the entire depth of the joint. As-built drawings indicate that this detail was changed to contraction joints with filler material in the top inch of the joint only. Subsequent repairs made by DWR resulted in additional filler material being placed deeper into the slab surface (see Appendix G). Once high velocity flows remove this material, what is left is a deep, open joint that short-circuits the flow path to the foundation.

Keyed joints in thin slabs are also discouraged in the 1960 version of "Design of Small Dams," because they can fail or spall during differential movement. Reclamation solved this problem by integrating the key and cutoff, using details like those shown in Figure E-1. Forensic investigation of the keys in the Oroville Dam service spillway chute (see Appendix D) showed that cracking, spalling, and delamination are associated with these keys. Combining this with the deep application of filler material at the repaired joints resulted in undesirable flow path conditions.

Another non-standard feature at Oroville was the sloping herringbone drainage pattern. This pattern was initially adopted following a BOC recommendation to adjust the angle of the drains based on the excavated foundation surface to achieve a 1% to 4% drainage slope to promote drainage, rather than being horizontal like other most spillways. The proposed variable slope recommendation would allow the drains to be placed near the foundation contact to maximize their effectiveness at removing ground water from the foundation surface. However, it appears from the surface cracking patterns above the drains, that the drains were installed at uniform spacing and slope angles in order to intercept the collector drain pipes at precise elevations and to avoid field adjustment of each individual herringbone drain. The original bid specifications drawings showed the drains being placed on backfill concrete when overexcavation occurred in the foundation, allowing the drains to remain at a uniform depth below the slab invert. This detail included 8-inch weep holes through the backfill concrete to connect the drainage to the foundation. This detail was adjusted following the BOC recommendations. The separate backfill concrete placements were eliminated by placing the drains on top of a formed gravel wall that elevated the drains so that they would remain within the slab with the top of the drain and gravel envelop combination typically placed within 7 inches of the top of the slab. It is believed this change made them more uniform and avoided field adjustments (see Appendix A for additional drain details). When overbreak of up to several feet occurred, instead of placing the drains on the formed gravel wall, they were raised up to the same position below the top of the slab by constructing the drains on gravel-filled sonotubes.

**Figure E-4: Photo Showing Plastic Sheeting Over Drains (Dated 9/30/1966)**

Compounding the drainage issue was the plastic sheeting placed over the drains, which would cause water leaking into the cracks above the drains to flow around the drains, rather than into the drains (see Figures E-2 and E-4). Clearly flow into the drains was expected to come from the foundation below, not the chute flow surface above. While it is understandable that the placement of concrete over unfiltered gravel drainage material could cause contamination, or plugging, of the gravel with concrete, the solution to place plastic over the drains also served to isolate the drains from leakage flows passing through the chute slab into the foundation. Many other spillways from that period used burlap to filter the concrete during placement. Burlap would still allow flow from open joints or cracks to enter the drainage system. At Oroville, the ability of flow entering the foundation through open cracks above the drains to enter the herringbone drains is restricted by the plastic membrane. Since there are no drains at the joints, unless a herringbone drain happens to cross the joint, water flowing into the open joints must travel along the foundation contact to reach a drain. These restrictions for flow from the chute entering the foundation could lead to a buildup of pressure at the foundation contact.

## 3.0 CURRENT DESIGN PRACTICES

Spillway design practices have evolved over time. The 1960 version of “Design of Small Dams” was readily available at the time the Oroville Dam service spillway chute was designed. EM 1110-2-2400 was new at the time the Oroville spillway was designed. Prior to that time, designers at Reclamation and others were using government publications such as “Treatise on Dams” [E-5].

While Reclamation has been using a risk-informed approach to design for decades, most other organizations have been slow to follow suit. “Best Practices in Dam and Levee Safety Risk Analysis,” which was first published in 2009, has made a risk-informed approach to spillway evaluation and design more accessible to others. Since the 1960s there are several design features and practices that have gained a broad acceptance, which were not necessarily being considered in the 1960s. A United States Society on Dams (USSD) 2006 paper by Trojanowski [E-6] discusses some of the historic advancements in spillway design over time.

In the late 1960s and early 1970s, more spillway chutes seemed to be designed with waterstops in the joints. Before that time, only crest or headworks structures and terminal structures seemed to consistently include waterstops. The use of waterstops began with installation of metal waterstops, and was changed to rubber waterstops, which had become more common in the 1960s. Rubber waterstops were more flexible than metal waterstops, which were more likely to be damaged by differential movements. However, metal waterstops seemed to be a better choice in hot climates where rubber-based materials degraded over a short time. By the end of the 1970s, PVC waterstops began replacing rubber waterstops. The PVC waterstops were being designed with a central bulb that could accommodate joint opening better than rubber waterstops.

Design details for spillways on rock foundations shown in the 1987 version of “Design of Small Dams” included two layers of reinforcement in chute slabs and keyed joints, with waterstops and continuous reinforcement or two layers of dowels. The waterstops at joints would prevent flow and pressure from entering the foundation. The two layers of reinforcement in spillway chutes helped reduce the opening of cracks and joints and offsets caused by differential movements.

In 1990, Reclamation provided guidelines for spillway chute cavitation in Engineering Monograph No. 42 [E-7]. This publication was in response to issues of minor and major cavitation damage that had occurred in the past. While designers had known about cavitation potential for a long time before this monograph was published, the monograph provided a more comprehensive understanding of the issues which could be incorporated in designs. Since cavitation damage could occur for both offsets into the flow, as well as offsets away from the flow, designers were not including an intentional offset at joints as often as they had in the past.

While the potential for uplift to increase as a result of offsets at open joints and cracks was understood by the 1960 Reclamation publication *Design of Small Dams*, other than providing offsets away from the flow at slab joints, there was no information related to the magnitude of the flow or uplift pressures that could be generated at these features. Reclamation published several papers about uplift related to stagnation pressures. The first paper on uplift caused by stagnation pressure was published in 1976 [E-8]. a second paper was published in 1988, following an incident at Big Sandy Dam spillway [E-9]. Trojanowski published two papers through USSD: the one in 2006 cited above and one in 2008 [E-10]. A laboratory study in 2006 resulted in a 2007 report by Reclamation [E-11], which was also presented in a 2008 USSD paper [E-12]. These documents suggested that water entering offset cracks or joints in spillway flow surfaces could generate high uplift pressures, which could result in chute slab failures. Had the staff responsible for inspecting Oroville spillway, and participating in the PFMA studies applied the information in these publications, and "Best Practices in Dam and Levee Safety Risk Analysis," they may have been able to recognize the potential failure mode before the February 7, 2017 failure of the spillway chute occurred.

In the 1960s, spillway designers had difficulty quantifying rock erosion in unlined spillway chutes. Most erosion studies related to soil or loose rock such as riprap. For example, Reclamation's *Engineering Monograph No. 25* [E-13] was first published in 1958. It includes a chart for sizing riprap based on bottom flow velocities, but has nothing on in-situ rock erosion. Current methods using "stream power" are available to designers today [E-14] to help predict rock erosion, but very little was available at the time Oroville spillways were designed.

The IFT was also provided with copies of some of the documents currently available to their staff. This included Reclamations Design Standards. Design Standard No. 14 applies to spillways, and Chapter 3 [E-15] in particular applies to the issues that have been discussed in this Appendix. Their staff also has copies of various USACE Engineering Monographs that are useful in spillway designs.

## 4.0 CONCLUSIONS

1. Almost all of the design features in the Oroville Dam service spillway chute design could be found at other dams designed and constructed during the same period around the 1960s. Even though the Oroville design did not have some of the features found in most of the spillways of that period, and had some features that were not common, it would be difficult to conclude that Oroville was not within the bounds of designs from that period. However, as with Oroville Dam, the service spillway was among the largest of its kind at the time, and it would be expected that would have been designed using the best practices at the time, rather than the typical practices, as seems to be the case.
2. Some of the features that were either missing or were additions to the Oroville spillway design may have been detrimental to the spillway, especially the large herringbone drains placed within the chute slab and the single layer of reinforcement.
3. Vulnerabilities included in the Oroville service spillway chute design were made more detrimental by changes to the drainage design and relaxation of the specified foundation excavation and cleanup requirements from those originally specified.
4. The lack of documentation related to design changes during construction makes it difficult to determine if the original designers were aware of the changes, or if those changes were made by others not familiar with spillway design practices. It is possible that poor communication during construction led to undesirable design features. While it is apparent

that DWR employed other organizations for assistance in their hydraulic designs and hydraulic model studies, there has been no indication that they reached out to others for help in understanding the best practices for spillway chute details at the time.

5. Although the spillway design was somewhat consistent with what other designers were doing in the 1960s, knowledge of developments since that time, related to spillway design and potential spillway chute failure modes could have helped identify problems in the spillway chute before a failure occurred.

## 5.0 REFERENCES

[E-1] Design of Small Dams – A Water Resources Technical Publication, United States Department of the Interior, Bureau of Reclamation, 1960.

[E-2] EM 1110-2-2400, Engineering and Design, Structural Design of Spillways and Outlet Works, Headquarters, Department of The Army, Office of the Chief of Engineers, November 2, 1964.

[E-3] Best Practices in Dam and Levee Safety Risk Analysis, A Joint Publication by U.S. Department of the Interior, Bureau of Reclamation, and U.S. Army Corps of Engineers.

[E-4] Interim Exploration Data, Oroville Dam Spillway, Office Memorandum A.L. O'Neill to D.P. Thayer, November 20, 1964.

[E-5] Treatise on Dams, US Department of the Interior, Bureau of Reclamation, Chapter 12 – Spillways, Design Supplement No. 2 to Part 2 – Engineering Design of Volume X – Design and Construction - Reclamation Manual, October 20, 1950.

[E-6] "Can Your Spillway Survive the Next Flood?" 26th Annual USSD 26th Annual Conference, San Antonio, TX, May 2006, John Trojanowski, US Bureau of Reclamation.

[E-7] Cavitation in Chutes and Spillways, Engineering Monograph No. 42, United States Department of the Interior, Bureau of Reclamation, April 1990.

[E-8] Johnson, Perry L, "Research into Uplift on Steep Chute Lateral Linings," Memorandum to the Open and Closed Conduit Systems Committee, Bureau of Reclamation, Denver CO, July 15, 1976.

[E-9] Hepler, Thomas E., and Perry L. Johnson, "Analysis of Spillway Failures by Uplift Pressure," Proceedings of the 1988 National Conference on Hydraulic Engineering, edited by Steven R. Abt and Johannes Gessler, ASCE, New York, NY: 857-866, 1988.

[E-10] "Practical Application of Research Related to High Velocity Flows Over Open Offset Joints in Spillways," 2008 USSD Annual Meeting and Conference, Portland, Oregon, on April 28-May 2, 2008., John Trojanowski, US Bureau of Reclamation.

[E-11] Frizell, K. Warren, "Uplift and Crack Flow Resulting from High Velocity Discharges Over Open Offset Joints – Laboratory Studies," Bureau of Reclamation, Dam Safety Technology Development Program, Report DSO-07-07, December 2007.

[E-12] “Recent Advances in Predicting Uplift and Structural Collapse on Spillways with Open Offset Joints or Cracks,” Warren Frizell and John Trojanowski, 2008 USSD Annual Meeting and Conference, Portland, Oregon, on April 28-May 2, 2008.

[E-13] Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators, Bureau of Reclamation, May 1984.

[E-14] “Scour Technology,” McGraw-Hill Education; 1 edition (December 6, 2005), by George W. Annadale.

[E-15] Design Standards No. 14, Appurtenant Structures for Dams, (Spillways and Outlet Works) Design Standard, Chapter 3: General Spillway Design Considerations, Final: Phase 4, August 2014.

# Appendix F

# Inspections and Reviews

**F1 – Reports Specific to Spillway Cracking and Drainage Observations**

**F2 – General Operational Reports, Reviews, and Assessments**

**F3 – Potential Failure Mode Analyses (PFMAs)**

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# Appendix F1

## Reports Specific to Spillway Cracking and Drainage Observations

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## TABLE OF CONTENTS

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## 1.0 INTRODUCTION

Due to the design of the drain system that interconnects the drains from immediately downstream from the headworks structure, the herringbone underdrains, and the lateral drains outside of the spillway walls, there are four possible contributors to observed flows:

- Seepage water from underneath the headworks (this mostly applies to the first drainage zone)
- Rainwater/runoff collected from outside the walls of the spillway
- Groundwater upwelling from hillside springs that were covered over by the spillway chute
- Water being injected into cracks and joints in the chute concrete slab
    - From gate leakage when the spillway is not in use but water is against the upstream sides of the gates
    - From flows down the chute during spills

Although there are multiple contributors, it is now readily apparent that the majority of the drain flows during spill operation are from water injection through the slab, and the IFT considers the observed large volume of flows to be unusual. A similar observation was made by the Board of Consultants (BOC) in their 2017 Report #1 [F1-1]:

> "The amount of drain water flowing from the pipe discharge openings along the spillway training walls seems extraordinarily large."

The extensive herringbone crack pattern above the underdrains is also considered to be quite unusual. This appendix documents available references to the spillway drain flows and herringbone cracks by the DWR and the FERC, to better understand what importance was placed on these two unusual observations, when the unusual nature of the observations became first thought of as normal (often referred to as "normalization of deviance," whereby departures from desirable conditions become expected and accepted [F1-2]), and how this continued until 2017.

The following report summaries are presented in chronological order.

## 2.0 DESIGN ENGINEER'S CRITERIA [F1-3]

There are only two sections dealing with the spillway chutes in these documents, as follows:

1. D1.3 Drainage System – All the concrete structures, flood control outlet, chute and ogee section of the emergency spillway have drains…The drains should be inspected yearly so that they do not become plugged. The 12-inch VCP risers along the spillway chute can be used to flush out the accumulated fines in the outfall and collector pipes. The drain system under the chute slab is interconnected from a collector pipe on one side to a collector on the other side of the chute walls.
2. D4.0 Overpour Section (Emergency Spillway) …The areas of maintenance to be checked include a yearly inspection of the underdrains to see that they are not plugged. A general inspection of all concrete should be made after each spillover.

Regular inspections of the drains immediately downstream from the emergency spillway have been undertaken, however the service spillway chute drainage has only been inspected as outlined in the following sections.

## 3.0 GATE COMMISSIONING

The IFT was unable to locate a commissioning report, and it is not known if one was prepared. Thus, there is no record available of any flows down the spillway chute during commissioning of the radial gates at the end of construction, although some wet testing would normally be undertaken in the commissioning of the new gates.

## 4.0 DSOD INSPECTIONS

### 4.1 <u>Spillway First Use Inspection Notes</u>

The spillway was opened under operating conditions for the first time on January 21, 1969, in response to inflows from a series of storms. Operation continued throughout February and part of March. The peak discharge exceeded 80,000 cfs, compared with a current estimated probable maximum flood discharge of 306,000 cfs, with all service spillway gates operating and the reservoir full before the flood. The first available recorded observations of spillway flow are documented in a series of notes dated 1/29, 1/30/69 [F1-4]:

> "Discharging an estimated 30,000 cfs at time of inspection. Hydraulic performance appeared satisfactory throughout. *A point of interest was the relatively heavy discharge of the drain outfalls.* The second from lower end on the right side was noted to be flowing half full. Mr. Clayton (of Oroville Operations Center) had logged the flow from each outfall on the previous Saturday and planned on doing it again. He will also remove the caps on several drain risers to check on conditions at that point. Mr. Clayton stated that experience with drain outfalls during rains and before spillway flows *indicates that the drainage is related to spillway flows*, although it is difficult to imagine how these sizable flows can leak through shrinkage cracks and joints in the concrete. It was suggested to Mr. Clayton that inquiry to the designers regarding this aspect of performance would be in order…The high flows from the spillway drains are mystifying but probably not dangerous as the chute is anchored. The chute will be inspected as soon as possible. [italics added for emphasis]"

These early records of drain flows were not located by the IFT. Although it was recommended to notify the designers, the situation had already been labeled by DSOD as "probably not dangerous." It is not known if the designers were notified.

By March 5, 1969, the DSOD inspectors were able to gain access into the chute and noted [F1-5]:

> "Cracks in the chute floor and walls were noted on about a 20-foot spacing apparently placing them on top of the underdrains. Some of these were open almost 1/16 inch possibly explaining the relatively high drainage at the outfalls….The

> flows from the chute drains discharging from the sidewalls were roughly estimated and are tabulated below from the top down:
>
> | | **Left** | **Right** |
> |---|---|---|
> | 1 | Dry | Dry |
> | 2 | Dry | 5-10 gpm |
> | 3 | Dry | 100+ |
> | 4 | 200- | 100+ |
> | 5 | 50+/- | 100 |
> | 6 | Moist | 50 |
> | 7 | 150+ | 25+/- |
> | 8 | 10-15 | 150 |
> | 9 | 150 | 50+ |
> | 10 | 200 | Dry |
> | 11 | 100 | 150 |
> | 12 | 50+ | 50+ |
>
> It was noted that the joint mastic was missing from many of the contraction joints but from none of the expansion joints. Mr. Clayton said he would recommend replacement of all missing joint mastic and patching of the more open cracks during the coming summer, but in general the condition of the spillway was agreed to be excellent and safe for continued use this season without repair."

These are the first records of individual spillway drain flows that were located for the IFT review. There is no mention by the DSOD inspectors regarding any follow up that Mr. Clayton may or may not have had with the designers prior to the agreement that the condition of the spillway was excellent.

### 4.2 Documentation Following First Use

Following the spillway use in January and February 1969, a special DWR performance report was prepared [F1-6]. This report includes a description of the chute underdrain system:

> "The chute is provided with underdrains on a herringbone pattern. The underdrains are spaced at 25 ft. on the flatter reach of the chute and 20 ft. on the steeper portion. Collector pipes at the sides of the chute gather the drainage from groups of herringbones and discharge into the chute. Twelve outfalls into the chute are provided through each wall."

Concrete cracking was summarized in the report, and the herringbone pattern above the underdrains was discussed in some detail:

> "Cracking of the chute concrete can be summarized as follows:
>
> 1. Wall cracks, normal to the invert, near most of the contraction joints. This cracking occurred shortly after construction

2. Transverse invert cracks between the contraction joints appear to be in plane of the herringbone underdrains. Only two of these have significant opening. This cracking also occurred during curing of the slab within a month after completion of slab placement…”

> “It is concluded that cracking of the chute was not caused by, nor worsened by, operation of the outlet. The herringbone drain lines reduce the effective invert slab section apparently more than do the ‘weakened plane’ contraction joints; thus they are the weak link and cracked during curing with shrinkage of the concrete.”

The “weakened plane” contraction joints referred to here are believed to be the intermediate contraction joints at 50-foot stationing. These “contraction joints” were formed by saw cutting or scoring the concrete surface after slip forming. Cracks above the herringbone drains provided stress relief on 20 or 25-foot centers. As a result, many of the “weakened plane” contraction joints never opened.

> “All observations lead to the conclusion that little or no water is seeping through the foundation of the headworks and that the main source of water in the chute underdrain system is the discharge within the chute. Factors leading up to this conclusion are:

1. No water appears to be coming from the foundation drain holes all during the period of operation. These holes are connected to pipes which discharge through the downstream face of the piers. No water was observed coming from these discharge pipes except when the radial gates were opened. This water came from the drains above the head seals which connect into the same piping system.
2. The quantity of water being discharged from the chute underdrain system seems to be dependent on the discharge rate in the chute. Three observations were made during the period of operation: once with a discharge of 17,000 cfs, with a discharge of 7,000 cfs, and after closing the gates. The approximate drainage ratios (calling 100 percent drainage at 17,000 cfs spillway discharge) are as follows: 7,000 cfs discharge, 70 percent; zero discharge (gate leakage only), 30 percent. Drainage quantities with an outlet discharge of 17,000 cfs are indicated on Figure 1.

> There is evidence that the drain pipes under the headworks, downstream of the foundation holes, are picking up seepage. …Even though the collectors discharge individually into the chute, they are perforated and communication between them is highly probable. Location of the discharge into the chute, therefore, is not necessarily indicative of the location of the source.”

## 4.3 IFT Comments on Documented Drain Flows

There are a total of 24 drain outfalls, one on each spillway side wall (labeled “L” and “R” for left and right, looking downstream) at 12 locations. Leakage quantity data from Figure 1 in the 1969 report discussed above are summarized in the following table prepared by the IFT, along with information from other sources as noted.

**Table F1-1: Leakage Data taken from 1969 DWR Report**

| Location | Stations taken from "As-built" drawing A-3B9-1 sheet 47 dated 9-24-69 | Stations taken from 2017 HDR draft report on underdrain system | Approximate Stations (Ft) (scaled from Figure 1, 1969 Spillway Performance Report) | Approximate Range of Leakage Quantity(GPM) Total of both outfalls |
|---|---|---|---|---|
| 1 | 15+00 | 19+59 | 19+60 | 0-10 |
| 2 | 17+00 | 21+71 | 21+65 | 0-10 |
| 3 | 21+00 | 24+66 | 24+60 | >100 |
| 4 | 25+00 | 26+34 | 26+35 | >100 |
| 5 | 27+00 | 28+10 | 28+15 | 10-100 |
| 6 | 29+00 | 29+60 | 29+70 | 0-10 |
| 7 | 31+00 | | 31+60 | 10-100 |
| 8 | 33+00 | | 33+60 | 10-100 |
| 9 | 35+00 | | 35+65 | 10-100 |
| 10 | 37+00 | | 37+65 | 10-100 |
| 11 | 39+00 | | 39+60 | >100 |
| 12 | 41+00 | | 40+60 | 10-100 |

These records are the only ones known to the IFT that document flows from the outfalls during spillway use. Note that Locations 1, 2, and 6 are recorded as having significantly less flows than all other outfall locations. These same three locations, along with 10R are also noted as having significantly less flows following closing of the gates. This could be due to partial obstructions of these drains from original construction, or an indication that there was less water injection, because the chute slab had less cracking and better quality jointing in these areas. Note that flow velocities at Locations 1 and 2 would be less than those further downstream, and lower velocities would produce less water injection, all other circumstances being the same.

The report clearly recognized that the substantial leakage quantities, as summarized in the table, originated from "discharge within the chute." However, there are no comments in the report on either the quantity or the cause of the leakage. The logical explanation that the seepage must be from water ingress to joints and cracks is not stated, and perhaps was not recognized. No record was found to indicate that the original designers, or geologists, were consulting and/or made any comment on these quite unusual observations of high quantity flows. Normalization of deviance had already begun at first spillway use.

## 5.0 INSPECTION NOTES, 1969 THROUGH 1976

A DSOD report from 4/22/69 [F1-7], by the same inspectors who made the January 1969 observations, notes that the Board of Consultants (BOC) members, Messrs. Leps, Sudman, and Turner were in attendance on 4/22/69. Also present were Mr. R. Wong of the Design Branch and Mr. W. Peak of the Geology Branch. There are two sentences regarding the spillway:

> "The headworks structure was visited and no distress noted. About 10,000 cfs was reported being discharged."

It is difficult to believe that the inspectors who found the drainage flows "mystifying" in January would not initiate a conversation on this performance with all the senior personnel onsite in April. All levels of DWR up to and including the Board of Consultants (BOC) were likely aware of the situation, and no one found the flows to be worth pursing further. However, the inspectors continued to be curious regarding the flows, further noting on August 8, 1969 [F1-8] that

> "Flows from the chute drains are drastically reduced with the smaller amount of water in the chute."

By 1971, the original DSOD inspectors had been replaced by other personnel, and reports again note the team's conclusion that the cracks in the spillway slabs were over the subdrains, where the slab thickness was least. There is little other comment. However, in a January 1973 report [F1-9] on the spillway chute, it is noted that:

> "The entire spillway chute along the area behind the left spillway wall was inspected. All conditions were normal."

Normalization of deviance in regard to the drain flows and the cracks was firmly entrenched. Interestingly, however, a 1976 inspection team included one person that had been involved with the first 1969 inspections that had found the drain flows to be "mystifying" [F1-4], so that loss of knowledge through personnel changes could not have been a major factor in this process.

The DSOD inspection report from January 14-16, 1976 [F1-10] contains the first reference found by the IFT regarding spalling:

> "Lateral cracking was observed between nearly every construction and expansion joint in the spillway, particularly in the steeper section. Occasional spalling has occurred along the cracks and also along both longitudinal and lateral joints.
>
> The spalling is generally minor, but in a few instances it is 3 or 4 inches deep, and in at least two instances the reinforcing steel is exposed. Some of the spalled areas along the construction joints obviously consist of patching material that did not have adequate bond. However, those areas along the lateral cracks have not been repaired previously.
>
> The more significant of the spalled areas should be patched, as velocities are high on the lower chute and the cavities could be deepened by cavitation action if not repaired. There is also the possibility that the water that can enter the subdrains could ultimately overtax it leading to uplift pressures, although there is no

indication of a problem of that type at the present time. Total outflow at the end of the chute, including gate leakage, subdrain flows and ground water intercepted at the walls is normal.

I will prepare a memorandum to Operations and Maintenance requesting that the spalled areas be scheduled for repair sometime in 1976."

## 6.0 RECORDS FROM 1976 TO 2002

The IFT requested DWR to search for any and all records pertaining to the spillway drains. A single, unreferenced data table for this time frame was provided to the IFT by DWR , as given below.

**Table F1-2: DWR Unreferenced Data Table**

| Inspection Date | Lake Elev. (feet) | 7-day Cumulative Precip(itation) (inch) | 30-day Cumulative Precip(itation) (inch) | Inspection Comments for Spillway Wall Drains |
|---|---|---|---|---|
| 2/18/1992 | 714.3 | 3.68 | 8 | Seeping |
| 1/15/1993 | 739.6 | 4.52 | 9.96 | Dry |
| 6/7/1993 | 899 | 1.4 | 3.04 | Each flowing 5-10 gpm |
| 8/17/1993 | 854 | 0.8 | 0.8 | Each flowing 2-5 gpm |
| 3/14/1994 | 832.8 | 0.32 | 2.84 | Flowing |
| 4/26/1994 | 838 | 0.96 | 1.84 | Flow significantly reduced |
| 3/10/1995 | 861.6 | 6.32 | 7.92 | Flowing at high rate |
| 4/12/1995 | 855.4 | 0.48 | 7.64 | Flowing |
| 6/20/1995 | 898.6 | 2.4 | 2.4 | Flowing |
| 2/27/1996 | 844.7 | 1.92 | 8.04 | Flowing |
| 6/14/1996 | 896.2 | 0 | 4.68 | Dry |
| 5/21/1997 | 879.7 | 0 | 0.84 | Dry |

The spillway was not in use during any of the noted periods. The above data are reproduced in Figure F1-1 below, to explore the possible influences of lake level and precipitation on drain flows. The preceding precipitation and lake levels are given for each subjective drain flow observation.

This graph shows no definitive relationship between overall flows and either precipitation or lake elevation. Although the graph shows that the largest flow is apparently recorded at the highest lake elevation, it also shows:

- Drains can flow at relatively low lake levels, possibly due to precipitation.
- Drains have been observed NOT to flow at high elevations or at high precipitation, whereas they have flowed at other times.

There is little other useful insight given by this relatively short, incomplete record. Who asked for these records to be taken (and with what reasoning) is unknown, but there apparently was no follow up until 2003.

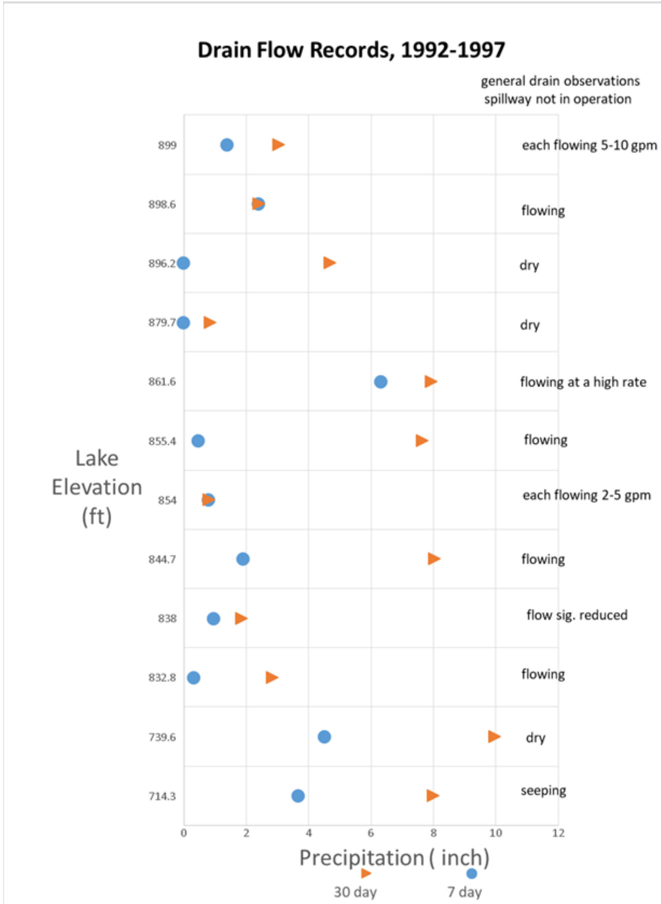



**Figure F1-1: Plot of Unreferenced Data**

The only other mentions regarding drain flows in this period were:

> The third FERC Part 12D Inspection Report from 1984 [F1-11] (See also Appendix F2):
>
> > "Most of the drains beyond about Station 25+00 along the right wall and several along the left wall were discharging significant amount of water."
>
> A decade later, the fifth FERC Part 12D Inspection Report (1994) [F1-12] simply states:
>
> > "Foundation pressures and drainage flows at the Oroville Dam facilities are consistent with past data."

However, it is not known whether spillway gates were open or closed at the time of the observations. It is also not known whether the second statement was intended to cover the service spillway chute drains. If so, it perpetuates the normalization of deviance.

No further comments regarding spillway drain observations were found until 2003.

## 7.0 2003 DRAIN REVIEW BY DWR

A reference to some drain flow estimations being documented in 2003 by Oroville Field Division personnel is given in annual reports as outlined in Appendix F2. This was indicated as being requested by an Engineer-in–Training in the Civil Maintenance Branch (later to become the Chief Dam Safety Engineer), who wished to better understand and evaluate the drainage system. In a series of emails [F1-13], the plans to monitor the drains while the reservoir level was expected to drop during the summer are given:

> "…you should conduct the spillway drain observations twice a week for the next the (sic) month and then cut back to once a week until the drains stop flowing."

> "…Water is flowing from drains located about 2/3 of the way up the sides of the spillway. The drain holes appear to be about 1/3 full of water….I believe the water is comming (sic) from a series of drains located beneath the spillway chute. That water is probably the result of leakage through the concrete joints…The concern is over how much flow there is, and wether (sic) it is normal or not. There don't seem to be any historical records on the subject. My recollection is that there is that (sic) the quantity of water comming (sic) out of the drains is normal."

> "During the inspection of the risers located along the right hand-side of the spillway chute….Nine of the thirteen 12" diameter vitrified clay pipe (VCP) has (sic) been vandalized…On June17 03 we started monitoring the outfall flows…Those flow data showed some of the outfall (sic), specifically the ones along the steep slope of the chute, to be in a dry condition despite the fact that the D/S and U/S outfall (sic) were flowing and the Lake elevation was as high as 899'… There is a possibility that some of the 12" Diameter perforated VCP collectors are partially or may be (sic) totally clogged at certain location (sic) due to the cobbles and dirt dropped into the risers or due to some other factors thus restricting or possibly preventing water from draining properly as originally designed."

In response to these observations, the risers were cleaned out by way of a specially constructed "grabber," and the casings were remediated, however, they were not probed or flushed. Records of drain observations were summarized in a later repor [F1-14] (see also Appendix F2) by one line as follows:

> "Typically, flows are seen from the drain holes when the reservoir elevation is above 825 ft."

A few files of unreferenced raw data were provided by DWR, and these data were summarized by the IFT in a series of graphs given in Figures F1-2 through F1-4. For these graphical representations and ease of interpretation, a logarithmic scale of flow in GPM has been used, with the terms "dry" and "trace or wet" being represented as 0.1 and 1 GPM, respectively. ">100GPM" has been arbitrarily plotted as 200 GPM.

Note that the spillway was NOT in use at any time during these observations, so that all flows would be due to gate leakage and precipitation only. It is evident from the IFT graphs given below that:

- There is very little or no flow (indicated by the “x”) from eight of the total 24 drain outfalls at full reservoir elevation of about 899 feet:

| Location | Approx. Station (ft) | Left | Right |
|---|---|---|---|
| 1 | 19+60 | X | X |
| 2 | 21+65 | X | X |
| 3 | 24+65 | X | |
| 7 | 31+60 | | X |
| 8 | 33+60 | X | |
| 10 | 37+65 | | X |

- The significantly lower flow at outfall location 6 that was noted on first spillway use in 1969 is not seen
- By the time the reservoir drops about 20 ft. to elevation ~ 880, flows from three more outfalls have either gone dry or are negligible:

| Location | Approx. Station (ft) | Left | Right |
|---|---|---|---|
| 6 | 29+70 | X | |
| 9 | 35+65 | X | |
| 11 | 39+6- | X | |

- By the time the reservoir is lowered another 10 ft. to about elevation 870, there are only four of the 24 outfalls still flowing, all at the furthest four downstream locations 8 through 12.

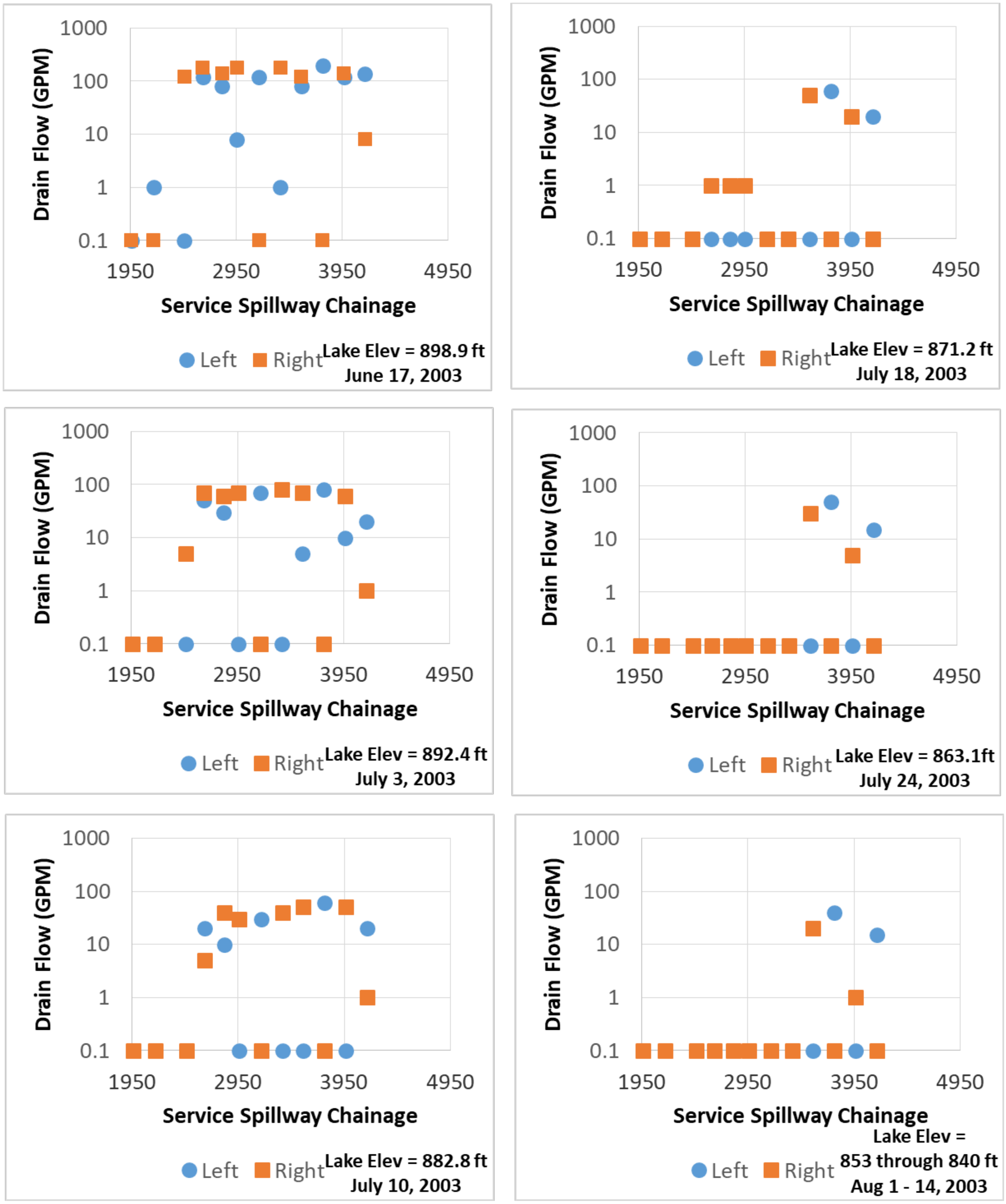



**Figure F1-2: Plot 1 of 2003 DWR Unreferenced Data**

Figure F1-3 shows the major effect of seasonal factors that could include thermal opening/closing of cracks, rainfall and runoff, etc. A series of three plots are shown over a period of a year, while at about the same reservoir elevation of 825 ft.: summer flows are negligible from all outfalls, while in winter, 18 of the outfalls are flowing, seven of these between 10 and 100 GPM each.

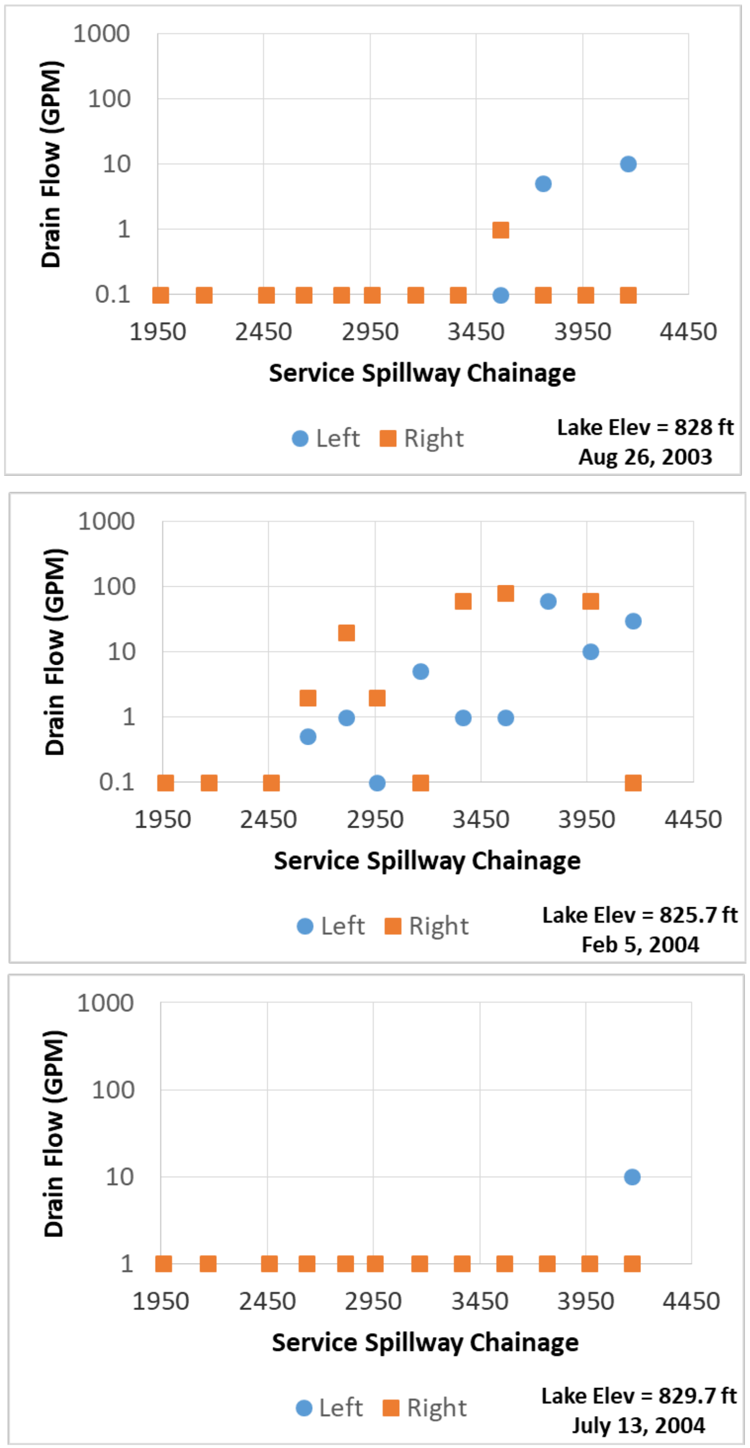



**Figure F1-3: Plot 2 of 2003 DWR Unreferenced Data**

Figure F1-4 illustrates the variable nature of the flows by comparing two plots separated in time by one week at the end of February 2003, with essentially no change in reservoir elevation. In this short timeframe the changes in flow are significant, and include:

- Outfalls 3L and 4L and 7R and 10R, start flowing up to > 10 GPM from being originally dry, while at the same time:
- Outfall 8L goes dry after originally flowing up to 100 GPM.

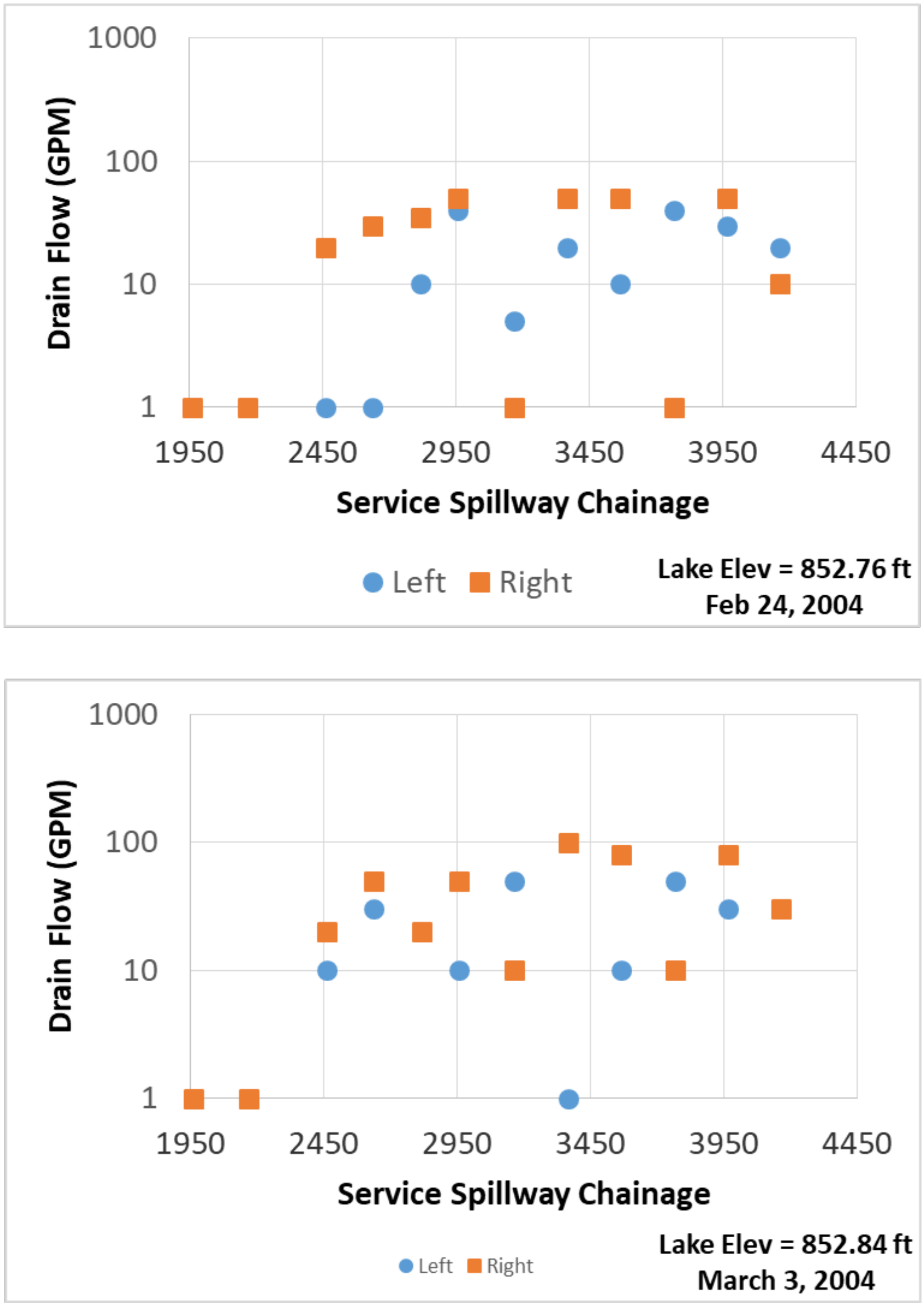



**Figure F1-4: Plot 3 of 2003 DWR Unreferenced Data**

In general, review of these records confirms that flows from the drain outfalls when the spillway is not in use are quite variable, dependent on a number of factors, and cannot be taken as a firm indication of changing drain efficiency or progressive bedrock erosion. One possible factor in the drain variability could be the temperature of the chute slab at the time of the reading, based on time of day and daily temperature fluctuations. However, no temperature data were provided.

To the IFT's knowledge, there have been no flow measurements/estimates since 2003. A general instruction was given by the Chief Dam Safety Engineer to verify that the drains were flowing during spill operations, however there are no records of these observations. The IFT was told during interviews that site staff were always asked whether the drains were flowing during spills, but results of these inspections were never recorded

It is unclear as to why the Civil Maintenance Branch of the DOE subsequently abandoned their research into the drainage system. This is further discussed in Appendix K1.

## 8.0 RECORDS FROM 2003 TO 2017

Since the 2003 observations, the only records of service spillway chute drain flows available are the occasional photograph, a number of which have been uploaded to the web since the incident. Due to either scale or photo quality (or both) it is generally very difficult to firmly establish the flow condition at the drain outfalls. However, a few photos given as Figures F1-5 through F1-10 do provide some insight:

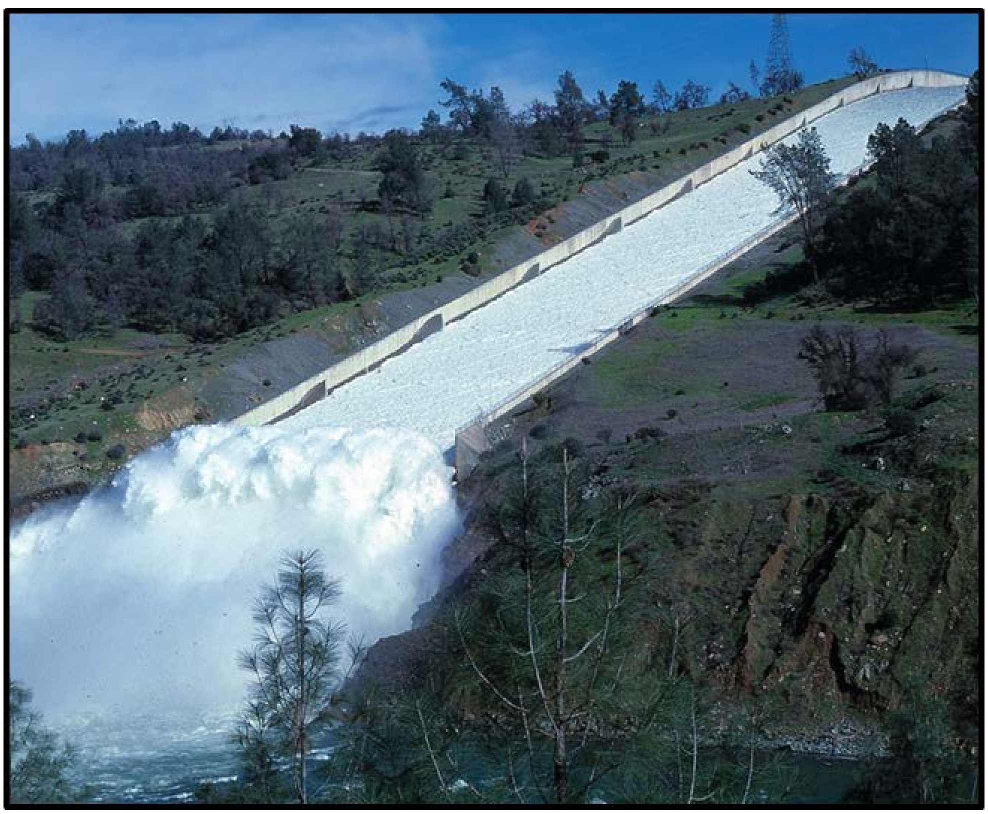
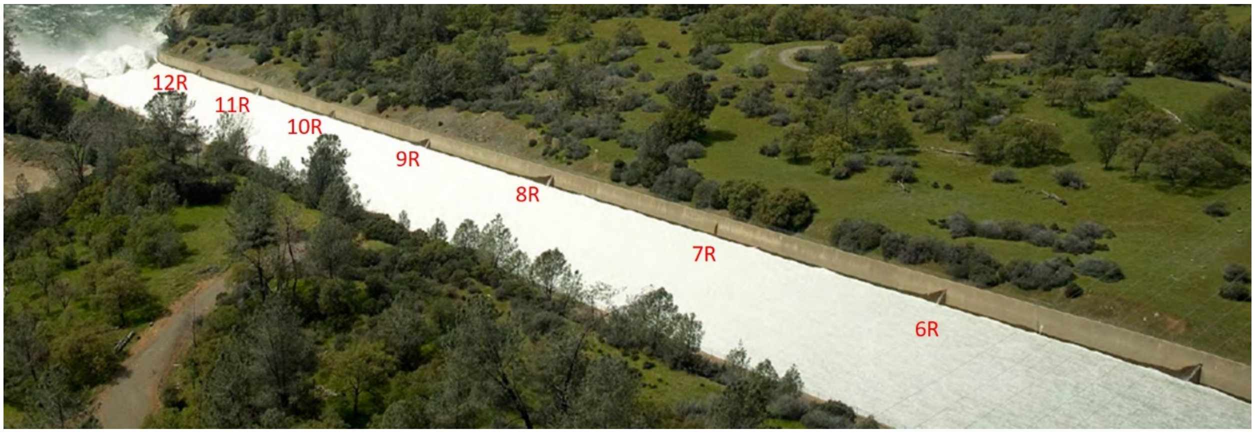
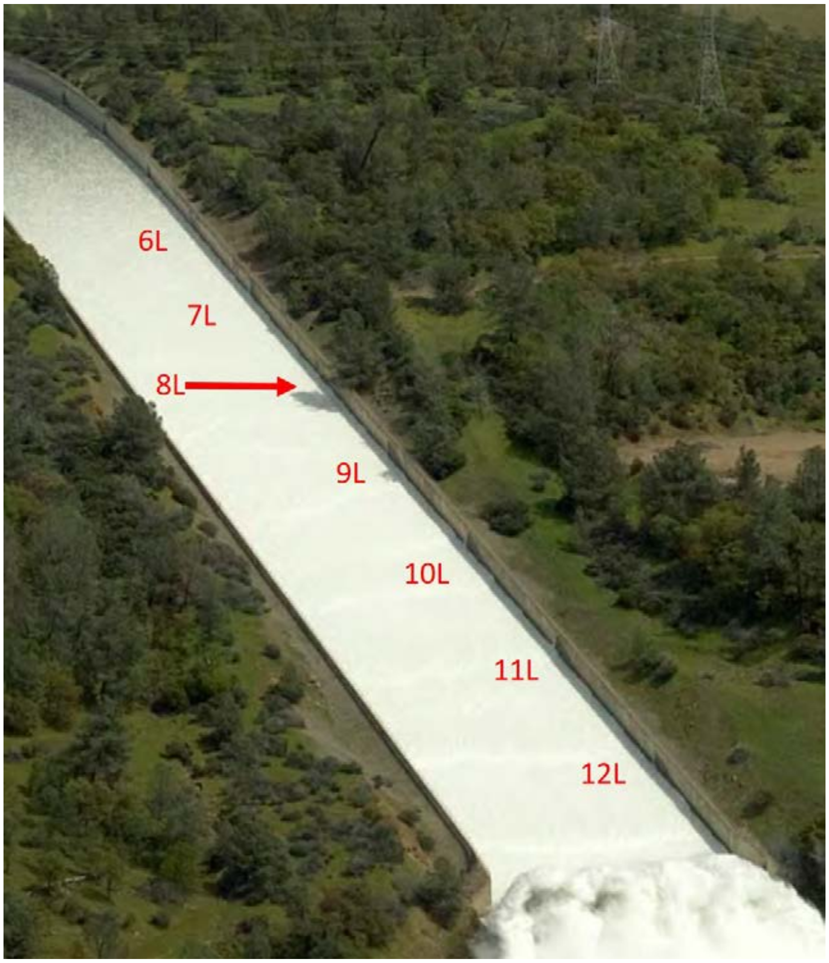


**Figure F1-5: February 3, 2006 (Photo provided by DWR): Spillway in use - all right side drains apparently flowing well. No photos available for left side.**


**Figure F1-6: May 17, 2006 - Spillway in use (photo provided by DWR): obscured view, but possibly little to no flow in outfalls 9L and 8L and possibly 10R.**


**Figure F1-7: November 9, 2007 [F1-15] – Spillway not in use: Outfall 7R is not flowing. However, Outfall 6R is strongly flowing, in contrast to notes during 1969 first use, where the total of both left and right outfalls at Location 6 was only 0-10 GPM total flow**




9L flowing

8L apparently not flowing

**Figures F1-8 and F1-9: April 2011, Spillway in operation (photos provided by DWR): Outfall 7R is only seeping compared to other outfalls.**


**Figure F1-10: January 27, 2017[F1-16] – Spillway not in use: Outfalls 7R and 8L are not flowing.**

## 9.0 FEBRUARY 2017 OBSERVATIONS

The IFT was contacted by an experienced hydroelectric operator from another utility who has observed the service spillway under flood conditions in previous decades. On an undefined date just prior to the incident with flow in the spillway, the operator remembers the overall flow from the drains "catching his eye" as he drove by the site. In general, he thought the flows, particularly in the area downstream from the eventual failure area, were significantly greater than he had previously witnessed. This observation however, cannot be verified.

## 10.0 OBSERVATIONS POST 2017 SPILLWAY INCIDENT

In late February 2017, and again near the end of March, qualitative monitoring of the flow from each of the 12 surviving outfalls at the upstream six locations was initiated following closure of the spillway gates, and the results are included in a post-incident report prepared for DWR [F1-17]. Similar to the 1969 observations:

> "Flow from the outfall drains appears to be related to, but not necessarily proportional to, flow quantity in the spillway."

One of the most significant post incident observations was that, with gates closed but leaking, the drains were flowing, but the drain flow stopped on the right side once the leakage from the gates along the spillway chute was diverted from that side by sand bags. This indicates that the source of the drain flow in this instance was water flow into the spillway slab joints and cracks. The photo below [F1-18], taken in this time period, also clearly shows spillway flow from leakage terminating at the construction joints, again indicating their open nature.


**Figure F1-11: Aerial View of Service Spillway following the Incident**

Following joint sealing work from February 27 to March 17, 2017 flow reductions varied from 0 to 80 percent with an average flow reduction of about 50 percent, again confirming the supposition that flows from the outfalls were related to water injection through the spillway chute. Occasional temporary turbidity was noted only at the upstream two outfall locations (1L and 1R, 2L and 2R), and was attributed to muddy road runoff that enters the drain system from outside of the chute walls during rain events.

## 11.0 RELEVANT DRAIN AND FLOW OBSERVATIONS DOCUMENTED IN ANNUAL REPORTS

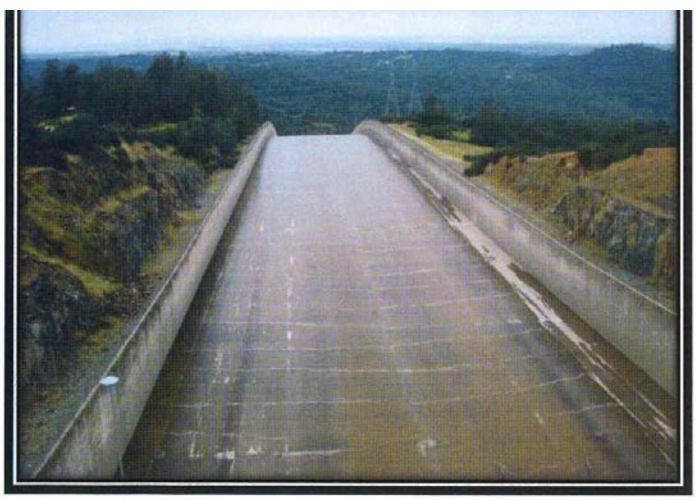
There are very few comments regarding drain flows in annual inspection reports. Access into the chute was deemed to be too dangerous at high reservoir levels due to the flow of leakage from the spillway gates on the chute surface. The IFT was told in interviews with Operations personnel that following a mandate from the FERC, the gate seals were replaced between 2009 to 2013, but the gates "just leaked worse." Thus, direct observation on the spillway chute was not undertaken when reservoir water was against the gates.

Because of the safety restrictions, observations of chute conditions were often made from a distance. Even then, the IFT document review shows that internal DWR inspectors, DSOD and FERC inspectors, as well as external reviewers, all noted the herringbone crack pattern coincident with the spillway chute underdrains. However, no-one considered these cracks to be of significance. For example: the Eighth FERC Part 12D Inspection Report (2010) [F1-19] states:

> "We observed some chevron cracking in the floor that is coincident with the underlying floor drains."

And a photograph from the 2010 annual FERC inspection report [F1-20] illustrates the crack pattern, as shown below:

"

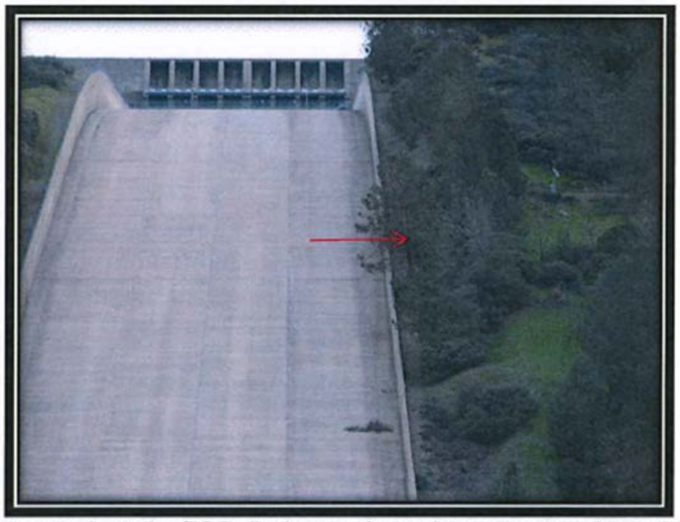


12. The concrete along the spillway chute has been repaired. The repaired herringbone crack pattern is said to reflect the underlying drain system. "

From the Feb 3, 2015 FERC Inspection [F1-21]:

> "The full length of the FCO discharge chute was inspected. Conditions appeared to be normal. The concrete repairs along the chute floor remain sound…. No signs of uplift were observed along the chute floor…A significant effort was made to clear

brush along the outside edge of the left chute wall. A lone tree, photograph 9, should also be removed.



9. This view looking upstream along the FCO discharge chute shows one tree (arrow) that needs to be removed following a significant effort to remove brush along the outside of the wall.

"

The tree noted is in the general area of the initial chute slab failure, adjacent to an outfall (8R) seen not to be flowing. As there has been much public speculation in regard to this particular tree, the IFT spent considerable time investigating the possibility of drain blockage at this particular location, and this is discussed in the following section. Refer to Appendix D for a general discussion regarding invasive tree roots.

## 12.0 IFT COMMENTS ON THE LONE TREE

The particular tree in question was closest to the left side of the spillway at about station 34+50. The IFT considered whether roots from this particular tree, only about 100 ft. or so from the initial failure area, could have been a contributing factor. The IFT believes that this hypothesis is unlikely, due to the layout of the drain system as illustrated in Figures F1-12 and F1-13:

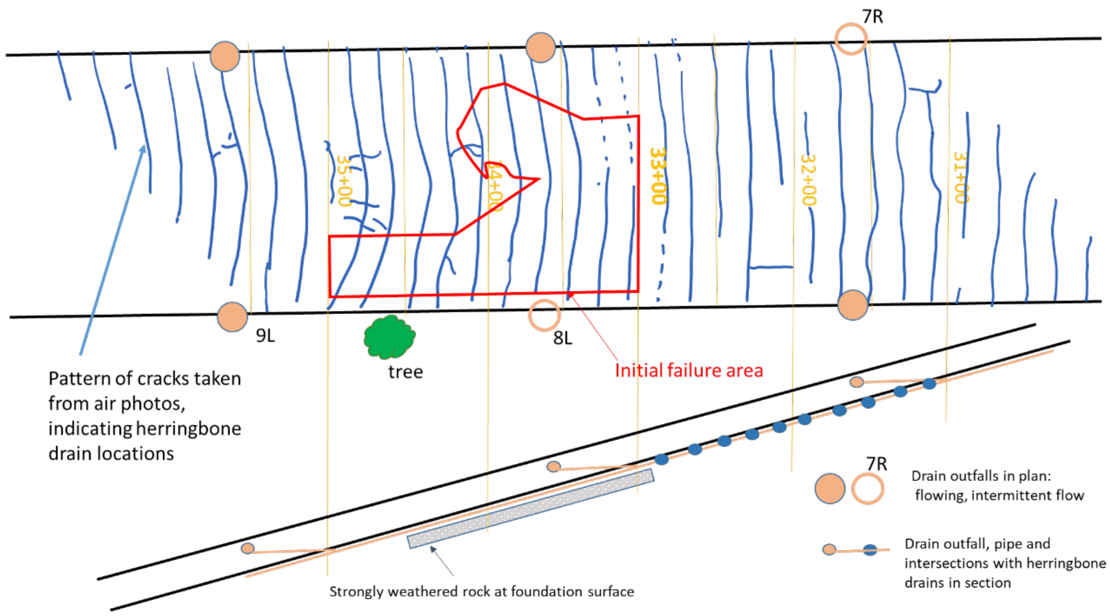



**Figure F1-12: Schematics of Spillway Chute near Initial Failure Area**

As shown in the plan views, the tree was located close to the left spillway wall, between Outfalls 8L and 9L, so that blockage by roots from this tree would affect Outfall 9L. Each outfall connects to the herringbone drains upstream of the outfall location, as shown in the profile view and lower plan view. The most downstream drain for each outfall is located about 60 to 70 ft. upstream from the outfall, and the most upstream drain is located about 260 ft. upstream of the outfall. Had roots from this particular tree entered the drainage system, they could have blocked the herringbone drains in the vicinity of the initial failure. However, the outfall for this section is at about station 35+65 (Location 9L), which in 2011 and 2017 photos is flowing freely. As such, roots from this particular tree could not have been significantly blocking the 9L outfall.

Outfall drain 8L, immediately adjacent to the initial failure area, has been observed not to flow, and thus was possibly blocked. This outfall collects flows from drains under the chute from about Station 31+00 to 33+00, upstream from the initial failure area as shown in the sketches. Blockage of 8L is a possible contributor to the failure under certain assumptions. If the slab was thick enough in this area (upstream from Sta. 33+00) that the drains were well above the foundation, and water leaking into the foundation could pass through a thick gravel layer placed beneath the drains where the foundation was overexcavated. This water could make its way to the downstream drainage set that is in the area of the initial damage. If this happened, the downstream drain set (9L), might not have had sufficient capacity to handle the combined drainage flow and caused flows to back up under the slab.

Another possibility possible explanation for no drain flow is that the water from the chute entered the foundation, and bypassed the herringbone drains by flowing under the slab through piping internal erosion channels downstream to the next drain set.

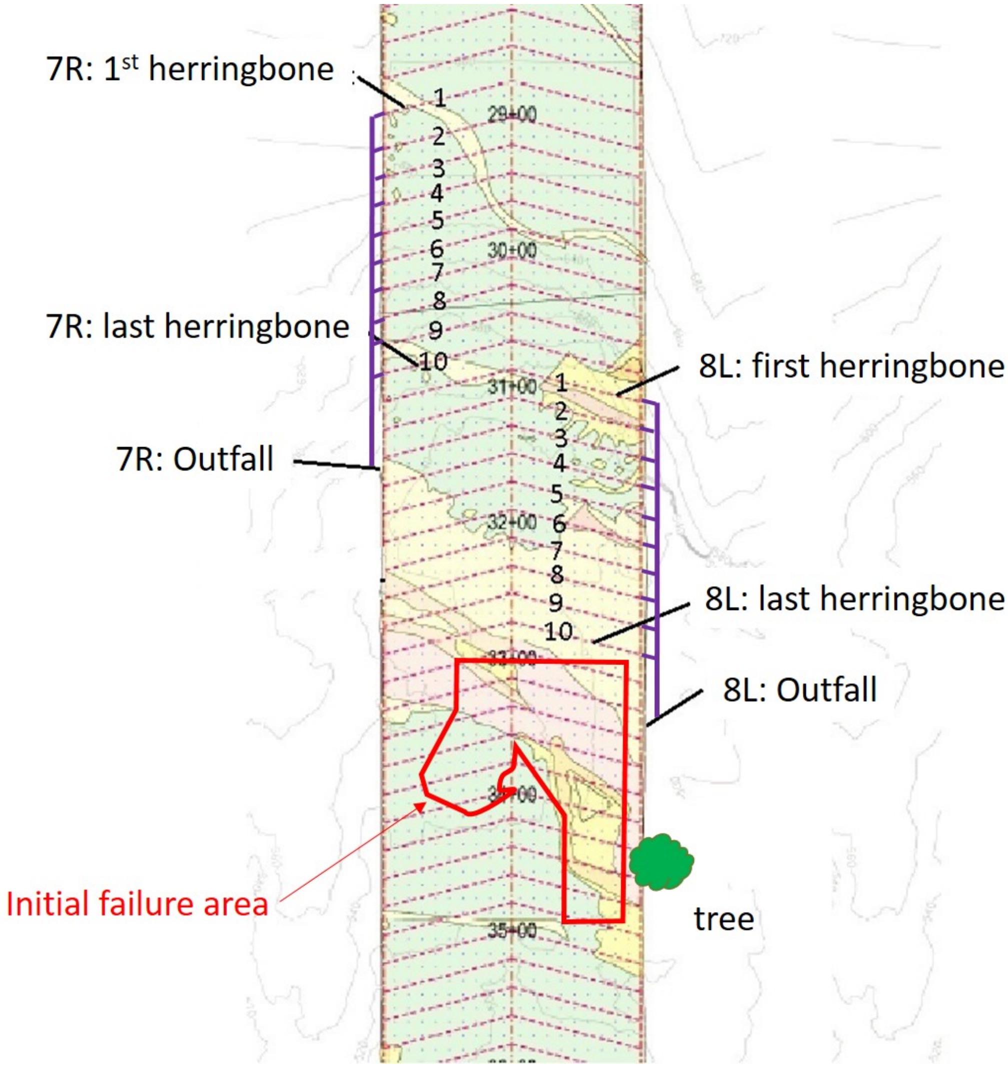



**Figure F1-13: Idealized Layout of Spillway Chute Underdrains upstream of Initial Failure Area (underlay of chute geology provided by DWR)**

However, it is also possible that either this drain was not connected to the outfall pipe prior to the damage, and flow from the herringbone drains discharged into the gravel outside the wall, or there was actually no inflow to the herringbone drains in that drain set because the repairs had not failed there, and no flow was entering the foundation.

Thus the lack of flow in the outfall pipe does not necessarily indicate that pressure was building under the chute slab in the zone immediately upstream from the initial failure. There is no further evidence to support either view.

## 13.0 IFT COMMENTS ON DRAIN FLOWS

In review of the available drain records as presented in the above sections, there are two significant findings:

1. Drain flows have been shown to be influenced by many factors; particular drains flow at some times, while not others. Thus, observations when the spillway is not in operation cannot be used to determine whether or not any particular drain outfall is plugged.
2. Outfalls 7R and 8L, two of the outfalls closest to the failure initiation zone, have been observed to not flow regularly when the spillway is in use since 2006, and this could be due to plugging by tree roots or a number of other mechanisms. However, the spillway has previously performed satisfactorily at higher flow volumes than that at the time of the chute failure on February 7, 2017, last previously in 2006. At most, the possible plugging of these drains could only be considered a contributing factor. However, reduced or no flows could also indicate that repairs within the chute along joints and herringbone cracks in these particular locations remained effective. It was noted that outfall 7R began flowing after the area of the chute downstream from this location had failed. At that time, it was reported that some of the joints in that drainage zone began to open. This opening is possible related to sliding of the chute slabs towards the open area downstream.

## 14.0 SUMMARY

The IFT considers both the extensive concrete cracking above the herringbone drains and the observed volume of flows from the chute drains during spillway operation to be highly unusual. The importance of these unusual observations, first noted in 1969, has been essentially ignored, and it took very little time for these observations to be "normalized."

### 14.1 Cracking in the Spillway

The extensive herringbone crack pattern above the underdrains was clearly documented and related to the relatively thin chute concrete slab section directly above the embedded drains from the time of first commissioning. Later internal DWR inspectors, DSOD and FERC inspectors, as well as external reviewers, all noted the herringbone crack pattern coincident with the spillway chute underdrains. The crack pattern is repeatedly considered to be "probably unimportant" by DWR. The FERC and external reviewers also simply noted the crack pattern in a number of their inspections, without further comment.

### 14.2 Flows When Spillway Is Not in Use

There have been sporadic observations of flows when the spillway gates were closed, with leakage down the spillway chute. In 2017, observations of reduction in drain flows following diversion of leakage flow along one side of the spillway, and subsequent joint sealing work, clearly indicate that the source of the drain flows from the outfalls is related to water injection through the spillway chute.

However, drain flows are far from uniform, with some outfall locations having significantly less flow than other outfall locations. This non-uniformity in drain flow has been noted in previous decades, at least in part as a function of lake elevation. In one series of observations:

- There is very little or no flow from eight of the total 24 drain outfalls at full reservoir elevation of about 899 feet; there is somewhat less flow at outfall 6L than at other flowing outfalls.
- By the time the reservoir drops about 20 ft. to elevation ~ 880, flows from three more outfalls have either gone dry or are negligible.
- By the time the reservoir is lowered another 10 ft. to about elevation 870, there are only four of the 24 outfalls still flowing, all at the furthest four downstream locations 8 through 12.

The largest flows are usually recorded at the highest lake elevation; however, this is not necessarily the case:

- Drains can flow at relatively low lake levels, due to precipitation.
- Drains have been observed NOT to flow at high elevations or at high precipitation, whereas they have flowed at other times.

Observations separated in time by one week at the end of February 2003, with essentially no change in reservoir elevation, show some outfalls start flowing up to > 10 GPM from being originally dry, while at the same time one outfall goes dry after originally flowing up to 100 GPM. Other observations show effects of seasonal factors that could include thermal opening/closing of cracks, rainfall and runoff, etc.

Thus, drain flows have been shown to be influenced by many factors; particular drains flow at some times, and not others. Observations when the spillway is not in operation cannot be used to determine whether or not any particular drain outfall is plugged.

### 14.3 Flows When Spillway Is in Use

High chute drain flows during first use of the spillway were documented in an operational report following the commissioning of the service spillway. It was concluded that "little or no water is seeping through the foundation of the headworks and that the main source of water in the chute underdrain system is the discharge within the chute." In addition, cracking above the herringbone drains was also identified as "possibly explaining the relatively high drainage at the outfalls."

Flows were not measured during use of the spillway gates since first use in 1969 at any time up until the chute failure in 2017. Following the chute failure, it was noted that "Flow from the outfall drains appears to be related to, but not necessarily proportional to, flow quantity in the spillway," similar to the 1969 conclusion.

Although drain flows caused by flow in the spillway chute were clearly recognized from 1969, and the logical explanation that the drain flow must be from water ingress through joints and cracks was identified, early inspectors noted that the flows were "mystifying." As some of these same inspectors were present during later external reviews, all levels of DWR up to and including the Board of Consultants were likely aware of the situation, however, no one found the flows to be worth pursing further. By the 1973 First 5-year FERC review, all conditions in the spillway were considered "normal."

The 1969 records of first spillway use indicated that the flows are not uniform. The first two outfall locations closest to the spillway gates, and outfalls 6R and 6L (near station 29+60) were recorded as having significantly less flow than all other outfall locations. This could be due to partial obstructions of the drains from original construction, or an indication that there was less water injection in these areas, perhaps because the chute slab had less cracking and better quality jointing. Later photo records show large flows from Outfalls 6L and 6R (as compared to 1969), but reduced flows in Outfalls 8L and 9L from 2006 onwards, and reduced flows from Outfall 7R from 2011.

## 14.4 Possible Reduction in Drain Efficiency

The photo records do show that a reduction in drainage efficiency close to the failure initiation zone is a possibility. However, if plugged or partially plugged, these outfalls would have been so over periods when the spillway has previously performed satisfactorily at higher flow volumes than in February 2017. Further, the non-flowing outfalls drained an area of the chute further upstream that was not part of the initial failure area. Although it is possible that there was blockage of these upstream drains, causing inflow to be diverted to the failed section downstream, there is no physical evidence that this actually happened. Thus, at most, the possible plugging of these drains could only be considered a contributing cause to the chute failure.

The IFT also considered whether roots from a prominent tree, only about 100 ft. or so from the initial failure area, could have been a contributing factor. The IFT believes that this hypothesis is unlikely, due to the layout of the drain system relative to the location of the tree.

Refer to Appendix D for further comment regarding tree roots.

## 15.0 REFERENCES

[F1-1] FERC Project No 2100- Oroville Emergency Recovery – Spillways, Independent Board of Consultants Memorandum No. 1, March 17, 2017.

[F1-2] Dam Safety Risk – From Deviance to Diligence, Steven, G. Vick, ASCE Geo-Risk 2017.

[F1-3] Design Engineer's Criteria for Operation and Maintenance, State Water Facilities, Oroville Division, Part II, Oroville Dam and Reservoir, October 1967.

[F1-4] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, J.F Chaimson and D.L. Christensen, 2/13/69. Date of inspection: 1/29, 1/30/69.

[F1-5] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, J.F Chaimson and D.L. Christensen, 3/10/69. Date of inspection: March 5,1969.

[F1-6] Oroville Dam and Lake, Performance of the Flood Control Outlet during the Storms of January-February 1969, DWR Division of Design and Construction, March 1969.

[F1-7] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, J.F Chaimson, 4/30/69. Date of inspection: April 22,1969.

[F1-8] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, J.F Chaimson, 8/21/69. Date of inspection: August 7 and 8, 1969.

[F1-9] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, G.H. Swanson, 1/30/73. Date of inspection: January 9 and 10, 1973.

[F1-10] Inspection of Dam and Reservoir in Approved Status, DSOD Inspection Notes, G.H. Kruse, DL Childress, DL Christensen and ED Stetson, 1/16/76. Date of inspection: January 14-16, 1976.

[F1-11] 1984 Inspection and Safety Review of Oroville-Thermalito Project Facilities, (Third Independent Safety Evaluation), Wahler Associates, June 1984.

[F1-12] Fifth Safety Inspection Report of the Oroville Dam Facilities, Woodward Clyde Consultants, May 1994.

[F1-13] Excerpts from emails between D. Panec and A. Samaan, and D. Panec and W. Cochran between June 23, 2003 and August 25, 2003.

[F1-14] Performance Report No. 11, August 2000 Through June 2004, Oroville Dam, DWR Division of Operations and Maintenance, August 2004.

[F1-15] photo on internet: http://www.panoramio.com/photo/19507969, dated November 9, 2007, (uploaded Feb 28, 2009).

[F1-16] photo on internet: http://photos.orovillemr.com/2017/01/31/photos-oroville-week-of-1-30-2017/#2

[F1-17] Oroville Dam Underdrain Inspection Report, DWR/Oroville FCO-01, HDR Project No. 10054687, May 6, 2017, *DRAFT*.

[F1-18] photo on internet: https://www.reddit.com/r/orovilledam/comments/5zrwrr/photo_oroville_emergency_spillway_aerial_march_8th/

[F1-19] Eighth Five-Year Part 12D Safety Inspection Report, Oroville Dam, GEI Consultants, January 2010.

[F1-20] FERC Inspection of Dam and Reservoir in Certified Status, W. Pennington,dated 6/1/10.

[F1-21] FERC Inspection of Dam and Reservoir in Certified Status, dated February 3, 2015.

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# Appendix F2

# General Operational Reports, Reviews, and Assessments

**TABLE OF CONTENTS**

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## 1.0 INTRODUCTION

As well as routine weekly and semi-annual facility inspections and reports over the decades, a number of overall facility reviews have been completed since construction by various parties. This appendix presents a summary of available reports reviewed by the IFT, which include reports in the following categories:

- FERC Part 12D Inspection Reports (completed at approximately five-year intervals)
- DWR Performance Reports (completed at irregular intervals)
- DWR Director's Safety Review Board (DSRB) Reports (completed at five-year intervals).

All of these above reports were prepared in response to federal and state regulatory requirements. A total of nine Part 12D Reports were written since construction, in compliance with FERC requirements. In addition, twelve DWR Performance Reports of a general nature, and four DWR Director's Safety Review Board reports, were prepared up to 2005. These DSRB reports were then discontinued as separate reports, and the DSRB requirements were covered through the ongoing series of FERC-mandated Part12D Inspection reports. The first FERC five-year inspection report discusses some design details that were not located elsewhere, including a reported uplift design criterion for the service spillway chute lining.

In addition to the above series of reports, three Potential Failure Mode Analysis (PFMA) reports were prepared in support of the FERC Part 12D reports. The PFMAs are related to the FERC Part 12D process, and are included as part of the Supporting Technical Information Document (STID) required for FERC-licensed hydropower projects. The IFT judged the PFMAs to be a sufficiently substantial effort to justify discussion in a separate appendix of this report (see Appendix F3).

Discussions covering the service spillway drains, and any measurements of flows from the drains, were not included in any of the above reports, except as noted in Appendix F1. Also covered in Appendix F1 are the IFT reviews of special spillway performance reports. Appendix C discusses the DWR emergency spillway reviews and assessments in 2005 and 2009, which focused on interpretation of spillway rock conditions. Appendix K2 discusses general regulatory aspects.

Table F2-1 presents a chronology of the reports discussed in this appendix.

The IFT notes that the intended scope of the DSRB reports was not well defined in the state regulations, and the scope of the DSRB work was generally determined by DWR with limited regulatory oversight by DSOD. The intended scope of the Part 12D inspection reports was better defined by FERC. However, in the opinion of the IFT, FERC's regulations and published guidance are somewhat ambiguous regarding whether the Part 12D reports are intended to be "updates" on changes since the prior five-year report, or relatively comprehensive reviews of the facilities which evaluate their design, construction, condition, and performance history relative to current states of practice. Appendices F3 and K2 provide further discussion on this topic.

**Table F2-1 : Summary of Operational Reports, Reviews and Assessments**

| FERC Part 12D Inspection Reports | DWR Performance Reports | DWR Director's Safety Review Board Reports | PFMAs (Appendix F3) | DWR spillway performance operational reports and reviews |
|---|---|---|---|---|
| | 1 - April 1968 | | | |
| | 2 - January 1969 | | | |
| | 3 - September 1969 | | | March 1969: Performance of the Flood Control Outlet during the Storms of January-February 1969 (see Appendix F1) |
| | | | | June 1969: Flood Control Outlet Performance Report #1 |
| | 4 - October 1970 | | | |
| | | | | |
| 1 - 1973 | 5 - October 1973 | | | |
| | 6 - April 1976 | | | |
| | 7 - April 1977 | | | |
| 2 - 1979 | 1977 to 1984: no reporting | | | |
| 3 - 1984 | | 1984 | | |
| | 8 - June 1987 | | | |
| 4 - 1989 | | 1989 | | |
| | 9 - December 1990 | | | |
| 5 - 1994 | | 1994 | | |
| 6 - 1999 | | 1999 | | |
| | 10 - July 2000 | | | |
| | 11 - June 2004 | Discontinued[1] | 2004 | |
| 7 - 2005 | 12 - July 2006 (draft copy) | | | Emergency spillway geology review (Appendix C) |
| 8 - 2010 | Discontinued | | 2009 (update) | Emergency spillway erosion assessment (Appendix C) |
| 9 - 2014 | | | 2014 | |

[1] The 2004 report was incorporated into the 2005 FERC Part 12D report.

## 2.0 JUNE 1969: FLOOD CONTROL OUTLET PERFORMANCE REPORT #1

This report is solely a record of hydraulic and structural instrumentation measurements since their installation. There are no further observations on the use of the spillway.

## 3.0 DWR PERFORMANCE REPORTS

These reports were produced by the DWR Division of Operations and Maintenance on an irregular basis, and the first four of these cover only the Oroville Dam itself. Report #5, October 1973, is the first report to cover the entire project, rather than only the embankment dam. However, the only reference to the spillways is in regard to instrumentation readings of uplift under the spillway concrete headworks structure. Reports #6 and #7 cover the period up to April 1977, following the general format of Report #5.

**Report #8** [F2-1] covers the period from 1984 to 1987**.** There is no comment or explanation for the gap in the record between 1977 and 1984. This report covers more subject material than prior reports, and it is the first report that provides a statement regarding the overall safety:

> "A thorough evaluation of the documented structural performance … and a review of the associated field inspection reports indicate the structures are functioning satisfactorily. Accordingly they are judged stable and safe for continued use."

It also summarized spillway releases made during this reporting period and other record years:

| "Spill Date | Max. Release cfs | Max. Inflow cfs | Total Release A.F. |
|---|---|---|---|
| 1/13/70 | 59,000 | 143,000 | 1,553,621 |
| 1/12/80 | 70,000 | 155,000 | 726,259 |
| 2/15/86 | 137,000 | 266,000 | 1,420,262 |
| 3/7/86 | 40,000 | 107,000 | 499,340" |

Comments on the spillway are no longer simply limited to instrumentation readings of pore pressure beneath the headworks structure, but include:

> "Repairs to Oroville Spillway concrete were completed in 1985. Pitting of the concrete spillway dentates was sustained during the record high releases in 1986. Although not structurally critical, the dentates will be cosmetically repaired (Item Complete)"

**Report #9** [F2-2] (1987 to 1990) is the first of these reports to include brief geological descriptions for the service spillway. These are taken from other report series and repeat the words:

> "Where fresh, the rock is hard, dense, greenish gray to black, fine to coarse grained and generally massive … The depth of weathering was found, during construction, to be generally substantial and varied greatly from place to place."

In terms of the spillway, it is noted that:

> "The only surveillance data being plotted for evaluation are the pore pressures under the Flood Control Outlet Structure."

There is no comment regarding whether or not it was satisfactory practice.

**Report #10** [F2-3] (1991 to 2000) repeats the same geological description from #9, and continues reporting on the spill history:

> "There have been 14 spills or releases through the Flood Control Outlets during this report period."

The two largest of these are given as:

| "Spill Date | Max. Release cfs | Max. Inflow cfs | Total Release A.F. |
|---|---|---|---|
| 3-09-95 | 80,000 | 140,647 | 1,234,672 |
| 12-27-96 | 130,000 | 276,578 | 2,013,300" |

This report is the first to summarize DWR responses to FERC and Director's Safety Review Board reports. For example, in response to a 1994 Director's Safety Review Board comments, they note that:

> "Repair of spalled areas in the spillway chute slab and energy dissipator dentates were made by contract in 1997 …"

This report is also the first to mention instrumentation other than for measuring uplift pressure directly under the headworks structure:

> "Sixty-eight piezometers were installed in the spillway gate bay walls. Forty-one are still considered to be functional. These instruments were only read during flood releases the first few years after completion of the Flood Control Outlet. Now they are all retired. The last reading was taken in 1982."

> "The Flood Control Outlet Structure has proven by past performance to be stable from uplift pressures, and since stable readings are becoming more difficult to obtain, these pore pressure cells should be abandoned."

**Report #11** [F2-4] (2000 to 2004) again repeats the general (and accurate) geological descriptions from previous reports. This report also describes the service spillway drainage system:

> "A series of pressure relief drains underlies the Oroville Dam spillway. Beginning in July 2003, Oroville Field Division personnel began estimating flows from the 12 spillway pressure relief drains when they are flowing. Typically, flows are seen from the drain holes when the reservoir elevation is above 825 ft."

As noted in Appendix F1, DWR was not able to locate some of these records for the IFT.

**Report #12** [F2-5], covering up to June 2006, again repeats the geological descriptions from previous reports, including the previous statement:

> "During construction, the depth of weathering was generally found to be substantial, and varied from place to place."

Three significant spills occurred during the report period:

| "Spill Date | Max. Release cfs | Max. Inflow cfs | Total Release A.F. |
|---|---|---|---|

| | | | |
|---|---|---|---|
| 2/27/04 | 6,844 | 25,021 | 26,500 |
| 12/22/05 | 75,000 | 158,507 | 1,305,062 |
| 2/27/06 | 33,000 | 79,334 | 394,496 |
| 4/3/06 | 19,000 | 52,677 | 858,224” |

The report repeats the short section describing the Spillway Pressure Relief Drain Holes from Report #11, adding one sentence, without further explanation:

> “No formal monitoring program is associated with these observations.”

This final report in the series also summarizes the PFMA work undertaken in 2004 and the subsequent 2005 FERC report and notes that:

> “The 2004 Director’s Safety Review Board Report is incorporated in the March 2005 FERC Part 12D report.”

The IFT was informed that both the separate Director’s Safety Review Board reporting and this series of 12 DWR reports were discontinued on the basis that the information in these reports would be covered by the ongoing series of FERC-mandated Part 12D Inspections.

## 4.0 FERC PART 12D INSPECTION REPORTS

As explained in DWR performance report #10 [F2-3]:

> “The Department is required by state law to have the performance of all state owned/operated dams reviewed at five-year intervals by a board of independent consultants … In addition, the FERC requires that a team of independent consultants conduct a five-year safety review of all dams holding a FERC license.”

There have been a series of these reviews to comply with FERC regulations. The reviews cover both the Oroville and Thermolito facilities. None of the reports cover detailed bedrock conditions, and they provide only general descriptions of the spillways, referring back to the Final Geological Report (see Appendix C) without further comment.

Almost all reviews commented on spillway capacity, and a few noted the necessity for proper gate operation and commented on the good reliability of the gate system, but none of the reviews questioned the capability of the chute to safely pass the flows.

**The first FERC five-year inspection report (1973)** [F2-6] is based on a July 10 through 14, 1972 inspection and was prepared by independent consultants. Some of the persons who provided various presentations to these reviewers were involved with the original construction, including J.W. Marlette who signed the Final Geologic Report for the Service Spillway. Comments in the 1973 report concerning the spillways include:

> “The natural slope below the spillway has not been altered, because operation of the spillway is unlikely. The spillways appeared to be conservatively designed and in good condition … It was reported that the Flood Control Outlet had passed up to 54,000 cfs on Jan 22, 1969. No appreciable scour from such flow could be seen …”

> "Under superflood conditions, the only real concern might be that the gates in the Flood Control Outlet are assuredly operable. To date, the gates have been both test operated (prior to acceptance) and service operated ('69 and '73) with reported ease."

In the description of the service spillway chute, it is stated that:

> "Expansion joints are provided in the walls and invert at intervals of 400 ft. Contraction joints are provided at 50 ft. intervals."
>
> "The invert slab is structurally independent from the chute walls. The slab is anchored to bedrock with grouted anchor bars, and provided with a system of underdrains. The 15 in. thick floor has sufficient weight to counteract uplift with a water surface at the top of the drain pipes. The anchor bars are capable of resisting an additional 5 ft. of uplift."
>
> "In review, *the combined spillways were judged to be competently designed and constructed*, to be in excellent condition, and to be structurally and hydraulically adequate to pass up to PMF flows." [italics added for emphasis]

Although specific comments and conclusions are presented regarding dam foundations, there are no comments on spillway foundations.

**The second FERC five-year inspection report (1979)** [F2-7] was prepared by one of the consultants from the first inspection, and reports on findings and significant events since his previous report. Comments relevant to the spillways include the following:

> "All major gates and valves except those of OROOUT (Low Level Outlet) were at least test operated annually. No substantial problems were noted."
>
> "In 1974, OROSP was in operation between Jan 15 and Jan 28, and between March 28 and April 11. The peak spill was about 30,000 cfs on Jan 18, 1974 … The second spill period noted had a peak spill of about 40,000 cfs."
>
> "In 1975, on Aug. 1 at 1:20 PM, an Earthquake of M 5.7 occurred about 7.5 mi. SW of ORO. Many aftershocks of considerable but less magnitude followed … No significant damage was observed …"
>
> "In 1977, repairs were made to OROSP chute invert concrete where damage, principally involving joint distress and minor spalling, had been experienced."
>
> "In 1980…it was necessary to operate OROSP…a peak spillway discharge of 70,000 cfs (the peak spill of record) was reached on Jan. 14….An inspection was made of OROSP by the Department on Jan 23, 1980. Only minor concrete erosion and plucking damage was reported. No significant bedrock erosion occurred in rock supporting the flip bucket."
>
> "OROSP was inspected on November 13 ... walking as far as possible down the spill chute to examine invert and wall concrete."

> "Examined local concrete invert repairs and sawed temperature stress relief joints for about 1400 ft. D.S. Appears OK."
>
> "Noted frequent *but probably unimportant cracks* in invert concrete over herringbone underdrains, and vertical cracks in some sidewall panels." [italics added for emphasis]
>
> "Near lower end, every alternate wall backfill drain discharging moderate amount."
>
> "Concluded Service Spillway to be in satisfactory condition. Auxillary Spillway never used and appear new. Can foresee no safety problem."

The report commented on the possibility of re-analyses resulting in an increase in PMF flow, and noted that the Emergency Spillway may need to be extended to provide the necessary extra capacity.

**The third FERC five-year inspection report (1984)** [F2-8] was prepared by a different consulting firm than the prior two reports. Comments relevant to the spillways (OROSP) include the following:

> "OROSP operated fifteen times since the most of those spills reported in Reference 5 (January 20, 1980) … All major gates and valves except those of OROOUT have been test operated at least annually."
>
> "Some slab concrete and joint sealant repair work was completed on OROSP since the previous safety review, and some remains to be completed this summer."
>
> "Due to gate leakage, the discharge channel was slippery and was not entered."
>
> "Periodic and detailed mapping of the condition of the floor panels is done by civil maintenance personnel. Special attention is given to the 1977 repairs of previously delaminated areas of the floor panel and the transverse joints which were modified by cutting the interpanel dowels and placing filler in the widened joints. The most recent survey was made on September 30, 1981. Areas affected are predominately narrow strips several feet long and about 2 inches deep along both transverse and longitudinal joints. The condition of these minimum 15-inch thick floor panels *anchored to sound rock* is not presently critical." [italics added for emphasis]
>
> "Most of the drains beyond about Station 25+00 along the right wall and several along the left wall were discharging significant amount of water. Flow from the invert drains at the terminal structure was small."
>
> "Periodic maintenance on the chute concrete has been done in a timely manner … The Department should continue to monitor OROSP slab condition and, where necessary, repair areas of drummy and spalling concrete."

**The fourth FERC five-year inspection report (1989)** [F2-9] was prepared by the same consulting firm as the prior third inspection. Comments relevant to the spillways include the following:

> "The flood of February, 1986 had no significant detrimental impact on the Oroville dam, its saddle dams, or its spillway."
>
> "Access to the spillway slab was prohibited, due to operating restrictions. The spillway dentate on the left side has eroded at one location to several inches deep, probably cavitation from the 1986 flood. Repairs to the spillway concrete slab were made in 1985. The Department conducted a detailed survey of the entire spillway slab in 1981 and 1987. Apparently, a number of drummy areas, mainly at patches and cracks, remain to be repaired, but no safety hazards are posed by these."
>
> "In 1986, a record flood inflow of 266,000 cfs occurred, and approximately 132,000 cfs was released through the spillway. This is nearly double the previous record release of 70,000 cfs in 1980. The spillway passed the recent large flow with only minimal isolated surface erosion."
>
> "Maintenance and operation procedures for Oroville Dam are, for the most part, excellent … The entire Oroville project is one of the most intensively reviewed and inspected projects known to the authors."
>
> "Repairs should be made to the eroded area in the spillway dentate. We concur with the recommendations of the report on the Independent Review of Safety (129) that small scale repairs of the spillway slab be performed on an as needed basis."

**The fifth FERC five-year inspection report (1994)** [F2-10] was prepared by a different consulting firm than the prior reports. Comments relevant to the spillways include the following:

> "At the flood control spillway, there are numerous locations of eroded and cracked concrete in the lower portion of the chute floor slab. Many of these are at locations which involved failure of previously applied inadequate concrete patches in the floor slab. A plan for repairing eroded areas and cracks and replacing all of the previously patched areas with competent patches should be developed and implemented prior to the start of the next rainy season. The plan should also include repair of chipped concrete areas on the small 'ears' at the upstream ends of the spillway dentates."
>
> "The Oroville Dam flood control and emergency spillways continue to be capable of safely handling the probable maximum flood (PMF) for the project. However, concrete patching is required at numerous locations in the flood control spillway chute, as discussed above."
>
> "The Oroville Dam facilities continue to be operated by DWR in a satisfactory manner, in terms of public safety. The facilities continue to be well maintained by DWR staff. However, the facilities are now 25 years old and will need an increasing level of maintenance as discussed above …"
>
> "Foundation pressures and drainage flows at the Oroville Dam facilities are consistent with past data. The recorded data indicate no cause for any safety concern"

This last statement quoted above is apparently a general statement for all facilities.

**The sixth FERC five-year inspection report (1999)** [F2-11] was prepared by a different team than the prior reports, comprised of two consulting firms. Comments relevant to the spillways include the following:

> "Overall, the flood control spillway was found to be in good condition. Gates 2 and 7 were opened during the field inspection; operation was smooth. March 1, 1999."
>
> "Deteriorated areas of the spillway chute slab noted in prior inspection reports were patched in 1997."
>
> "Current maintenance and surveillance appear adequate for the Oroville project."

**The seventh FERC five-year inspection report (2005**) [F2-12] was prepared by a different team than the prior reports, comprised of two consulting firms. Comments relevant to the spillways include the following:

> "A new feature of the Commission's Dam Safety Program is the Dam Safety Performance Monitoring Program (DSPMP). An important part of the DSPMP is the Potential Failure Modes Analysis (PFMA). The PFMA process formally identifies potential failure modes of the structure, components, or modes of reservoir operation that could lead to uncontrolled release of the reservoir. The Independent Consultant then considers the identified potential failure modes, and evaluates current monitoring and surveillance programs, and their adequacy to provide early detection of performance indicative of such failure modes."
>
> "Only one potential failure mode (PFM) was identified by workshop participants; it was rated Category IV (ruled out). The PFM was related to the piping of core material through broken instrumentation tubes. The detailed PFM report is presented in the Supporting Technical Information Document (STID)."
>
> "DWR's operations and maintenance procedures for Oroville Dam are adequate … No operational and maintenance PFM's were identified in the PFMA report."
>
> "The peak spillway discharge in the last five-year period occurred in February and March of 2004, with a flow of 6,844 cfs."

**The eighth FERC five-year inspection report (2010)** [F2-13] was prepared by a different consulting firm than the prior reports.

This report again reviewed the 2004 PFMA study, and the effort included a one-day PFMA update workshop (see Appendix F3). Comments relevant to the spillways include the following:

> "Operation and maintenance for the project is being performed to an acceptable level."
>
> "We recommend an evaluation of the adequacy of the spillway to pass the design flood event (PMF) be presented in a single volume report … This should be completed by January 2014."

> "Oroville Dam Spillway Inspection and Condition Assessment: … In April 2008, personnel from DWR's O&M headquarters and Oroville Field Division inspected the concrete surfaces of the spillway. The spillway was evaluated visually for cracks, erosive wear and spalling and by sounding using dragged chains and hammers for incipient spalls and voids. Overall, the condition of the spillway was found to be good. The defects tended to be previously repaired crack systems where prior patchwork is failing. The predominant failure mechanism for the concrete panels appeared to be freeze/thaw, exfoliation along construction joints, or stress crack systems. Many of the voids and hollows that were detected during the inspection were incipient spalls near the surface of the concrete panels. Little, if any, panel deformation (heave/settlement) was noted. Likewise, significant joint expansion and contraction was not observed. There were some signs of superficial erosion on the corners of the energy dissipaters (dentates) and some broken edges at several locations along the flip bucket end of the spillway … DWR indicates that the spillway repairs were completed in December 2009."

> "The February 1986 storm produced … a maximum outflow of 137,000 cfs. On January 1, 1997 … the peak outflow was 160,000 cfs … In addition, significant flood control releases were made in water year 2006 … Peak spillway discharge during the past five years … 75,000 cfs. The spillway has not operated since 2006."

> "In 2003, DWR updated the probable maximum flood (PMF) using PMP estimates from HMR59."

> "[At] the Flood Control Outlet spillway … A contractor was marking repair areas…during the inspection…The chute floor was in generally satisfactory condition considering the intended repairs …We observed some chevron cracking in the floor that is coincident with the underlying floor drains …We were informed the wall drains had been recently cleaned. The wall drains are inspected annually and cleaned as necessary …"

> "The emergency spillway was viewed from the Flood Control Outlet hoist deck … Soil cover over bedrock appeared to be on the order of about 1 to 4 feet."

In an overview of the geology of the dam area, it is stated that "The amphibolite bedrock has been described as sound, hard rock."

**The ninth FERC five-year inspection report (2014)** [F2-14] was prepared by a different team than the prior reports, comprised of four consulting firms. This report summarizes the PFMA workshop conducted in August 2014, which was a two-week process facilitated by a consulting firm which was not involved in any prior reviews for Oroville Dam. Comments relevant to the spillways include the following:

> "The dam is being well operated by DWR and project structures generally appear adequately maintained … all operation and maintenance (O&M) is being performed to an acceptable level."

> "Conclusions regarding the PMFA Report: Lessons learned in conducting PFMA workshops since 2004 were incorporated in the 2014 sessions, which the Board concludes to have been adequate and comprehensive across the whole range of viable PFM's identified for Oroville Dam."

> "R6: The Board recommends a thorough review of the stability calculations for the Emergency Spillway Structure … including current PMF and seismic loadings."

> "Peak spillway discharge during the past 5 years occurred on March 20, 2011 … which consisted of 31,500 cfs through the FCO (Flood Control Outlet)."

> "Sections 3.1 and 3.2, detail the outcome of the 2014 PFMA workshop, which is considered by the Board to have been thorough, systematic, and comprehensive."

The report details seepage measurements at a number of locations at various facilities, but does not mention the outflows along the service spillway (FCO) chute. The report indicates the following regarding the condition of the service spillway chute:

> "The spillway chute floor is in good condition (Photo 14). The spillway walls are in satisfactory condition, with no significant concrete distress, scour, or horizontal deflections ... The wall drains are inspected annually and cleaned as necessary. Repairs performed on concrete erosion on the detents and flip bucket lip were in good condition …"

The report repeats the sentence from the Eighth Part 12D report regarding the emergency spillway area:

> "Soil cover over bedrock appears to be on the order of about 1 to 4 feet."

but it does note that:

> "In the event that the Emergency Spillway is called into use, significant erosion with considerable downstream debris may occur, which will contribute to the already-expected flood flows and debris associated with a major flood flow over the spillway."

The report describes the foundation of the spillway concrete headworks structure in some detail, but does not mention the chute foundation:

> "The Board notes that the construction documentation (as-built geologic maps in References 4 and 5) of the dam foundation and spillway foundations is of high quality and detailed and, together with the excellent foundation treatment and comprehensive grouting program, make it highly unlikely that any undetected, significant potentially adverse structures (shears or faults) traverse the dam foundation and spillway foundations."

The report also presented findings to comply with California regulations requiring an Independent Director's Safety Review Board on a five-year cycle. Seven questions were asked of the Board, who supplied answers and recommendations. These questions and answers/recommendations were of a more general nature, and did not include any mention of either spillway that pertains to the IFT's investigation.

The IFT notes that the report contains design drawings for the spillway chute slab details in its Appendix C (Project Figures), and a diligent review of these drawings in comparison with the state of practice as of 2014 would have revealed a few of the deficiencies in the chute slab design, as described in various sections of this IFT report. The IFT did not find these chute slab details included in the 2014 PFMA report, the STID for the 2014 report, nor in any other prior Part 12D or PFMA reports. The IFT also did not find evidence that these chute slab details were evaluated as part of the 2014 Part 12D inspection and report preparation, which leaves the unanswered question of why these drawings were selected for inclusion in the 2014 Part 12D report, yet not in any other reports.

## 5.0 DWR DIRECTOR'S SAFETY REVIEW BOARD (DSRB) INDEPENDENT SAFETY REVIEW REPORTS

Prior to the DSRB reviews being merged with the FERC Part 12D process, there appear to have been four of these DSRB reports, as noted below.

**1984:** The IFT found references to this report (believed to be the first in the series), however a copy of the report was not located.

**1989** [F2-15]:

> "An earlier review was made in 1984 by a previous Board whose recommendations have since been acted upon… The 1984 Board recommended that the Department continue to monitor the spillway slab conditions and, where necessary, repair areas of drummy and spalling concrete….Some spalls were repaired in 1985….the record peak spillway release of 137,000 cfs made in 1986, did not result in any apparent change in the spillway slab condition…A detailed condition survey…was made in…September 1987… The present Board suggests that small-scale repairs, on an as needed basis, are more cost effective than deferred maintenance which can result in more costly repairs…""In general, the Board finds all dams and appurtenances to be safe for continued operation…The Board found the Department personnel to have full knowledge about the dams and related structures and to be aware of their responsibilities."
>
> "Specifically the Board recommends….3. Continued condition surveys of the spillway slab with early repair on an as needed basis."

**1994** [F2-16]

> "The Oroville Dam and its appurtenances are considered by the Board to be satisfactory for continued safe operation….The Board recommends that: …periodic surveys of the condition of the spillway be continued and that repairs to damaged areas be promptly addressed…"
>
> "3.2 Repairs to Spillway Slab : The 1989 Board recommended that the condition of the spillway slab be assessed and that needed repairs be made promptly. This

> recommendation has been completed and repairs to most areas of drummy concrete have been completed, but have not been entirely effective. The present Board examined the spillway slab and concludes that the post-1989 repairs have been partially effective. Although areas of repair that have failed do not represent a threat to satisfactory spillway operation, they should be repaired again in order to preclude additional progressive damage. It is suggested that the Department consider recent advances in concrete repair technology in making the repairs."

**1999** [F2-17]:

This report is in similar format to the 1994 report, with a similar general assurance statement. No recommendations are given regarding service spillway chute condition. There is no specific mention of the service spillway chute, other than to note DWR's response to the 1994 recommendations:

> "Repair of spalled areas in the spillway chute slab and energy dissipator dentates were made by contract in 1997. The Department will continue to inspect the spillway chute and make any needed repairs: The Board concurs that the Department has complied with the recommendation."

Spillway comments were focused on recommendations to update the PMF study, and in regard to strengthening the end pier anchor rod system, investigating the cause of roadway bridge concrete spalling etc.

## 6.0 SUMMARY

None of the reports covering the Oroville assets that were prepared for regulatory purposes identified the design inadequacies and unusual observations that would have forewarned of the service spillway chute slab failure and the emergency spillway performance issues in February 2017. The reports were not adequate in many aspects:

- There was cutting and pasting of accurate geological descriptions from previous reports, including statements such as "The depth of weathering was found, during construction, to be generally substantial and varied greatly from place to place." This was done without commenting that such descriptions directly contradicted statements made in the DWR 2005 and 2009 bedrock erosion documents (Appendix C), and other statements in the regulatory documents themselves.
- Inaccurate characterization of sound rock conditions over the entire areas of the spillways was perpetuated by truncating earlier, more complete descriptions. For example, the 8th FERC Part 12D report made this statement: "The amphibolite bedrock has been described as sound, hard rock." The 1960s introductory words "Where unweathered…" were omitted. Although attributed to the dam foundation, this impression would be easily transcribed to the spillways by a reviewer of this report.
- Inaccurate descriptions of ground conditions downstream from the emergency spillway were repeated as if they had been actually observed: "… was viewed from the Flood

Control Outlet hoist deck … Soil cover over bedrock appeared to be on the order of about 1 to 4 feet." (From both the $8^{th}$ and $9^{th}$ FERC Part 12D inspection reports.)

- It was noted that there is no formal monitoring program for the service spillway drains, without further comment regarding whether or not this would be satisfactory practice.
- Reference was made to the presence of the cracking coincident with the chute underdrains (as late as 2014), but no concerns were raised.
- There was focus on the *capacity* of the spillway to adequately pass the Probable Maximum Flood (three major PMF studies were performed in the life of the project: the original design study, a 1980 update, and another update in 2003), without questioning the *capability* of the structures themselves to do so.
- There were requests for spillway chute repair works in many locations noted to be previously repaired, and these repairs subsequently were deemed to be complete and adequate in later reports, although the repairs were obviously not robust and were prone to repeated failure.

The IFT believes that these reports are not necessarily deficient in view of similar reports in the industry; rather, they are likely a fair representation of such reports. "Normalization of deviance" (where departures from desirable conditions become expected and accepted) was being perpetuated by blindly building on previous reports. In general, full reliance was being placed on these reports for their due diligence regarding dam safety, and internal reviews or work beyond that mandated by FERC was not being considered necessary by many owners. This was "business as usual," and future failures to detect significant flaws, leading to future incidents such as that at Oroville, must be expected unless there are significant changes in industry practice. Also, over the time these reports were prepared, there were significant changes in the understanding of spillway failure modes that were not mentioned by any of the reviewers.

The IFT believes that comprehensive and detailed reviews of older designs against current best practices for design and construction are required. DSRB and FERC Part 12D regulatory requirements appear to be ambiguous regarding how comprehensive their reviews are intended to be. This ambiguity gives some flexibility to dam owners and regulators, however the IFT believes that it has also contributed to many reviews not being as comprehensive as was intended and needed. Going forward, the IFT believes that the effort to accomplish such relatively comprehensive reviews will be significantly greater than what has generally been done in past five-year FERC Part 12D Inspections, and changes to the structure of the Part 12D requirements may also be needed. This is discussed further in Appendices F3 and K2.

## 7.0 REFERENCES

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[F2-2] Oroville Dam, Performance Report No. 9, July 1987 - December 1990, DWR, Division of Operations and Maintenance, January 1, 1991

[F2-3] Oroville Dam, Performance Report No. 10, January 1991 Through July 2000, DWR, Division of Operations and Maintenance, August 2000

[F2-4] Oroville Dam, Performance Report No. 11, August 2000 Through June 2004, DWR, Division of Operations and Maintenance, August 2004 (unsigned final draft)

[F2-5] Oroville Dam, Performance Report No. 12, July 1, 2004 Through June 30, 2006, DWR, Division of Operations and Maintenance, December 2006 (unsigned final draft)

[F2-6] Inspection and Review of Oroville-Thermalito Project Facilities for State of California Department of Water Resources, W.L. Chadwick and L.M. Lepps, February 1973

[F2-7] 1979 Inspection and Review of Safety of Oroville-Thermalito Project Facilities (Second Five-Year Review), Thomas M. Leps, Inc., February 1980

[F2-8] 1984 Inspection and Safety Review, Oroville-Thermalito Project Facilities (Third Independent Safety Evaluation), Wahler Associates, June 1984

[F2-9] 1989 Inspection and Safety Review (Fourth Independent Safety Evaluation), Oroville Dam and Bidwell Canyon and Parish Camp Saddle Dams, Wahler Associates, November 1989

[F2-10] Fifth Safety Inspection Report of the Oroville Dam Facilities, Woodward Clyde Consultants, May 1994

[F2-11] Sixth Part 12 Safety Inspection Report for the Oroville Dam, Gomez and Sullivan Engineers and Findlay Engineering, September 1999

[F2-12] Oroville Dam FERC Part 12 Report, prepared by HDR and GeoMatrix, March 2005

[F2-13] Eighth Five-Year Part 12D Safety Inspection Report, Oroville Dam, GEI Consultants, January 2010

[F2-14] Ninth Five Year Part 12D Safety Inspection Report / 2014 Director's Safety Review Board, Oroville Dam, prepared by Rizzo Associates, URS, AMEC, and GeoSystems, December 2014

[F2-15] Independent Review of Safety of Oroville Dam, No. I-48, by Board of Consultants D.D. Campbell, P.L. Cole, J. Sciandrone (chair), March 1989

[F2-16] Independent Review of Safety of Oroville Dam and Associated Dams and Structures, by Board of Consultants D.D. Campbell, P.L. Cole, R. James, May 1994

[F2-17] Independent Review of Safety of Oroville Dam, Thermalito Diversion, Forebay and Afterbay Dams, Feather River Fish Barrier Dam, by Board of Consultants D.E. Bowes, A.B. Arnold, N.F. Parrett, Chairman, May 1999

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# Appendix F3

# Potential Failure Mode Analyses (PFMAs)

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## 1.0 INTRODUCTION

In the early 2000s, a Potential Failure Mode Analysis (PFMA) was developed for each FERC-regulated project in conjunction with the regularly scheduled Part 12D 5-year inspections. For subsequent Part 12D 5-year inspections, the initial PFMA was reviewed and updated, as judged appropriate by the Part 12D Independent Consultant. In recent years, FERC has judged that the original PFMA reports were not fulfilling the agency's goals, and therefore many PFMAs, including that for the Oroville project, were completed anew, rather than simply being updated. Although the PFMA process is related to the Part 12D process, the IFT judged it to be a sufficiently substantial effort to justify separate discussion in this appendix.

The dam safety community places great confidence in the PFMA process, with the expectation that it will provide a strong basis for helping to ensure the safe retention and passage of reservoir water. However, the most recent PFMA for Oroville Dam, despite being considered by those involved as thorough and complete, did not identify the various inadequacies that led to the February 2017 spillway incident.

This appendix provides a background to the PFMA process and evaluates the Oroville Dam PFMAs conducted in 2004, 2009, and 2014, with emphasis on the 2014 PFMA, since that was a much more extensive effort than the prior PFMAs. This appendix also presents general comments by the IFT regarding the PFMA process, and argues that a critical review of the process is warranted.

## 2.0 BACKGROUND TO POTENTIAL FAILURE MODE ANALYSES (PFMAs)

In 1980, the United States Department of the Interior, Bureau of Reclamation (Reclamation) first published the manual *Safety Evaluation of Existing Dams* (SEED). Reclamation's SEED program instituted formal examination teams, representing a wide range of disciplines, to comprehensively review the design, construction, and performance history of all Reclamation dams. This was, and continues to be, an excellent guide for the dam safety community. From the 1983 edition [F3-1]:

- "… the formal examination is to be characterized by an emphasis on a fresh look at the safety of the dam and appurtenant features and a comparison of the dam against state-of-the-art standards for design, construction, performance, and safety evaluation procedures."
- "The Team has the responsibility to review thoroughly all data and information, including that obtained during the onsite examination, relative to the safety of the dam."
- "Engineering data and records originating during the construction period should be reviewed to determine if the structures were constructed as designed or that the necessary design revisions were made for any unusual or unanticipated conditions encountered."
- "The intent of analyzing all available design, construction, and performance records is to become fully acquainted with the physical features and performance history of

the dam and appurtenances and identify any design, construction, performance, or operational deficiencies."

- "If new engineering theories or improved analytical methods are available, investigation of the original design of the dam utilizing contemporary theory and analyses should be considered."
- "Team members should be suspicious, inquisitive, noncomplacent, methodical, and possess sound engineering judgment. They should have an independence of view and avoid quick, unfounded conclusions."
- "The members of an Examination Team must be aware of the modes of dam failures. To understand and identify the potential failure of or weaknesses in a dam, Team members must have extensive knowledge of the causes of failure. Research and study of previous failures are required for Team members to reinforce their engineering understanding of how and why failures occur. The Team members should use all sources available to them for obtaining reports and descriptions of failures."

The SEED process is thorough and intended to provide a comprehensive review, and the SEED Manual lists and discusses numerous specific failure modes, as well as policies and procedures for technical analyses of identified failure modes. However, the SEED Manual does not present any structured or formalized methods for identification and screening of potential failure modes, leaving it to the team to identify and apply "state-of-the-art standards" for "safety evaluation procedures," as noted above.

During the past several decades, a variety of structured and formalized methods for identification of potential failure modes have been developed in various industries. For example, by 1985, Failure Modes and Effects Analyses (FMEA), and related failure mode identification and criticality analysis techniques, were well established in various disciplines in a number of industries, including the nuclear, aerospace, and petrochemical industries. In part due to its structured framework and necessary auditability, FMEA is incorporated into a number of national and international standards, including those by the US Department of Defense and the North American automotive industry. From *Risk and Uncertainty in Dam Safety* [F3-2]:

> "FMEA techniques are structured, logical frameworks that allow informed operatives and specialists to use available knowledge and information in a systematic way to lead to an understanding of the sources of risk in the system. The key benefits of the technique are transparency and amenability to audit … It also provides the facility to allow third parties (perhaps without specialist engineering backgrounds) to review the process and, if necessary, to question the inputs and outputs."
>
> "The primary skill required is to understand the functional nature of the system being analysed. This requires knowledge of how the system was designed and built, how system function is achieved, and how it has been and is being operated. This

> gives a basic appreciation of the potential weaknesses within the system and forms the basis for carrying out a failure mode-type analysis."

From British and European standards [F3-3]:

> "Where levels of risk are identified in industrial facilities and systems, the inductive approach is preferred and therefore the FMEA is an essential design tool. It should, however, be supplemented by other methods, particularly where multiple failures and sequential effects must be studied … FMEA proves to be essential but not sufficient."

In a FMEA, a failure mode is defined as simply "the manner in which an item fails" [F3-3] or "the manner in which a component, system or subsystem could potentially fail to meet the design intent" [F3-4]. In general, in a FMEA, a system (operational aspects as well as physical attributes) is broken down from its highest level of functioning into subsystems, which are in turn broken down to basic elements, resulting in an explicit model of the system being developed. Potential modes of failure for each of these elements are then identified, and the effect of the failure of each element on the subsystem is also evaluated, and so-forth upwards along numerous paths until the effect on overall system functioning is known. The severity of each elemental failure is classified, and probability of occurrence is estimated.

In the 1980s, Reclamation adapted and simplified FMEA for use in its SEED evaluations. Later, by the late 1990s, Reclamation had incorporated evaluation of failure modes into its risk-informed decision-making process as Potential Failure Modes Analysis (PFMA), and others in the dam safety community were investigating the adoption of failure mode analyses. *Risk and Uncertainty in Dam Safety* [F3-2] was being written to present the culmination of thinking up to that time, and it included discussion of FMEAs in some detail. The book was in part intended to provide dam owners with a basis for the establishment of risk management practices, and to provide a basis for what is now referred to as risk-informed decision-making. The book was the result of an international joint sponsorship by nine dam owners, including the US Army Corps of Engineers and Reclamation, under the Dam Safety Interest Group of CEATI (see Appendix K1). Contributions to the book were made by some of the sponsors and others prominent in the emergent practice of risk assessment for dams. It was noted at the time that:

> "The [FMEA] technique may require a great deal of time and a very significant resource commitment … While the basic technique is valid for dams, certain modifications may be required to apply the technique to dam safety practice."

Early drafts of the book were available to all of the sponsors, contributors, and reviewers. By the end of 2002, with the assistance of one of the reviewers of the draft, FERC had developed their own simplified variant of the technique, and labelled it Potential Failure Modes Analysis (PFMA). From the FERC website [F3-5]:

> "December 4, 2002: As part of the Dam Safety Performance Monitoring Program (DSPMP) … FERC developed guidance for carrying out a potential failure mode analysis … Draft Chapter 14 of the Engineering Guidelines describes the goals and expectations in the conduct of a PFMA and also presents a known, workable

> procedure for the conduct of the potential failure mode analysis. Those procedures … have been used and tested during the pilot project phase of the DSPMP."

During the period that *Risk and Uncertainty in Dam Safety* [F3-2] was being finalized for publication, a January 2004 workshop was held in San Francisco by FERC, which reviewed the results of early applications of the PFMA processes and how to improve them. The first Oroville Dam PFMA was held later in 2004, the same year as publication of the above noted book, which covered FMEAs, but did not include the PFMA process as developed by FERC.

Reclamation and the U.S. Army Corps of Engineers (USACE) both use PFMA as the initial step in their risk-informed decision-making processes, leading to subsequent quantitative risk analyses or semi-quantitative risk analyses. FERC adopted PFMA as a standalone process and has been using it for all of its licensed hydropower dams for about 15 years, and FERC has incorporated PFMA as part of its recently issued risk-informed decision-making (RIDM) guidelines. PFMA has also been adopted by some owners and practitioners in the international dam safety community. However, to the IFT's knowledge, use of PFMA remains limited to the dam safety community.

The PFMA process is considerably less structured than the FMEA techniques practiced in other industries, and has also often not provided the level of comprehensive review intended with the SEED process.

In the FERC process [F3-6], a PFMA team generally includes the following members, with the intention of bringing diverse perspectives into the process: a team leader designated by the dam owner, at least one facilitator, at least one individual familiar with the facility, at least one external independent consultant (including Independent Consultant(s) involved in the Part 12D inspection), and at least one participant from FERC. Some members of the team are considered to be "core team" members, and additional "participants," observers, and support staff may also be present.

The PFMA process can be summarized as involving the following steps [F3-6]: (a) assembling the team, (b) collection of available background information, including geologic data, historic records and photos, prior engineering analyses, inspection reports, monitoring and instrumentation data, and operational and performance data, (c) site review and inspection, (d) thorough review of the available information, (e) workshop sessions, which involve open-minded team "brainstorming" to identify potential failure modes (PFMs) (each defined as a chain of events) and their potential consequences, followed by evaluation of these potential failure modes, (f) identification of opportunities for surveillance, monitoring, and/or risk reduction for identified potential failure modes, and (g) documentation of the PFMA. The goal, at least in principle, is to be thorough and to identify *all* potential failure modes, considering a variety of loading conditions.

In evaluating potential failure modes, the team brainstorms to identify factors which make each potential failure mode more or less likely, and more or less consequential. A process such as voting or discussion to reach consensus is then used by the team to place each potential failure mode in one of four categories, as described in Table 1 below, which is taken from the current version of the FERC Guidelines [F3-6]. Potential failure modes placed in Category IV are ruled out from further consideration.

**Table 1 - Categories of Identified Potential Failure Modes**

| | |
|---|---|
| *Category I -* | *<u>Highlighted Potential Failure Modes</u> - Those potential failure modes of greatest significance considering need for awareness, potential for occurrence, magnitude of consequence and likelihood of adverse response (physical possibility is evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.* |
| *Category II -* | *<u>Potential Failure Modes Considered but not Highlighted</u> - These are judged to be of lesser significance and likelihood.* Note that even though these potential failure modes are considered less significant than Category I they are all also described and included with reasons for and against the occurrence of the potential failure mode. The reason for the lesser significance is noted and summarized in the documentation report or notes. |
| *Category III -* | *More <u>Information or Analyses are needed in order to classify</u> these potential failure modes to some degree lacked information to allow a confident judgment of significance and thus a dam safety investigative action or analyses can be recommended. Because action is required before resolution the need for this action may also be highlighted.* |
| *Category IV -* | *<u>Potential Failure Mode Ruled Out</u> Potential failure modes may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the potential failure mode, or the potential failure mode is clearly so remote a possibility as to be non-credible or not reasonable to postulate.* |

From FERC website [F3-7]:

> "… PFMA is an *informal* identification and examination of "potential" failure modes for an existing dam …" (italics added for emphasis)
>
> "The concept is to focus on identified, targeted areas of potentially serious and more-likely dam safety deficiencies so that limited financial resources can be used most effectively in ensuring dam safety and public safety."

The focus is on investigating those potential failure modes considered by engineering judgment to be most likely detrimental to public safety, and categorizing or ruling out others on the basis of the above categorization table.

## 3.0 2004 PFMA

This first PFMA for Oroville Dam, conducted in 2004, was a one-day workshop with a total of 33 team members [F3-8]. These team members included eight members of the "core team," including a consultant facilitator, two external consultants, a DWR DOE geologist, a DWR DOE engineer, a DWR O&M operations person, a DSOD engineer, and a FERC engineer; fifteen other "participants," included representatives from DWR (including O&M headquarters, OFD, and

DOE), an external consultant, five "observers" from DWR, the Los Angeles Department of Water, another organization, and four support staff from DWR.

The 2004 PFMA report [F3-8] notes that:

> "Prior to the workshop, the Independent Consultants and DWR representatives gathered background documentation including previous safety inspection reports, stability analyses, hydrologic analyses, geologic and seismic information, construction history, and surveillance data for Oroville Dam."

From this description, it is not clear whether similar background information was gathered for the spillways, as compared to the main dam. Also, given the large volume of information related to the design and construction of Oroville Dam, it is unclear how it was determined which of the documents were most important for the team to review.

The results of this PFMA which are most relevant to the IFT's investigation can be summarized by the following excerpts [F3-8]:

- "Major floods up to PMF can be passed through Oroville spillway successfully without significant concern relative to debris blockage, power supply, some gate malfunctions or delays in releases … with recognition that the emergency spillway will kick in once the reservoir elevation exceeds 901 …"
- "Only one Potential Failure Mode was identified … related to leakage into broken instrumentation tubes introducing reservoir pressure to the core [of Oroville Dam] … After discussion … it was recognized by the PFMA team that this potential failure mode was probably not credible but it was carried forward …" [this potential failure mode was ultimately placed in Category IV, based on the FERC classification]
- "*Although it was already well known that the foundation rock for these structures is strong*, it was surprising to see the hundreds of shears shown on the detailed foundation geology maps (See Fig 2-34)." [italics added for emphasis; the referenced figure is for the Oroville embankment dam, and spillway foundation drawings are not referenced]
- "A large amount of soil, rock and trees will be washed into the Feather River if the emergency spillway is utilized. It is unknown if this debris will make it to Thermalito Diversion Dam and cause any adverse effects on flow passage or tailwater. From observations it appears that significant adverse effects are unlikely."

Of note are the words italicized above – there was apparently no discussion regarding the assumption of good quality foundation rock, regardless of the "surprising" findings upon review of the foundation mapping.

For the one potential failure mode which was identified and developed, two "risk reduction actions" were identified. In addition to this potential failure mode, four additional "candidate" potential failure modes were identified, but were found to be physically impossible, not "reasonably possible," or unsupported by physical evidence, and, therefore, were not carried forward for further evaluation. For the candidate potential failure modes which were not carried forward, thirteen "risk reduction actions" were identified. None of the candidate potential failure modes were related to the spillways.

## 4.0 2009 PFMA UPDATE

An update to the 2004 PFMA was performed in 2009. This was a one-day workshop with a total of 18 team members [F3-9]. These team members included a DWR facilitator, three external consultants, and fourteen other participants, including representatives from DWR (including O&M headquarters, OFD, and DOE, including two DOE geologists), DSOD, FERC, and an external consultant.

The results of this 2009 update which are most relevant to the IFT's investigation are summarized by the following excerpts [F3-9]:

- "As part of the 2010 Eighth Part 12D safety review, a one-day workshop (covering all the Oroville-Thermalito Project) was conducted to review the PFM's developed during the 2004 PFMA workshops to determine if they remain appropriate or if there are any that have been omitted or reclassified … an additional Potential Failure Mode for Oroville was proposed and discussed. The PFM related to the failure of the Palermo Tunnel Outlet … but rated as Category IV (ruled out)."
- "The channel below the emergency spillway is heavily vegetated with brush and trees. The participants of the 2009 PFMA Update workshop discussed that if [the] emergency spillway were to operate, the vegetation below the channel would likely be uprooted and may accumulate at the Thermalito Diversion Dam ... *This was not necessarily considered a dam safety issue requiring risk-reduction measures, but it emphasizes the importance of proper operation of the spillway gates and releases, both at Oroville Dam and Thermalito Diversion Dam, to avoid problems at even lesser flows*." [italics added for emphasis]
- "There is substantial spillway capacity through the emergency spillway even if all gates are closed. The spillway rating curve shows that the emergency spillway has a capacity of 445,000 cfs at Elev. 920 feet. With zero freeboard to the nominal crest of the dam (Elev. 922 feet) it is estimated that the emergency spillway can pass 520,000 cfs."

In addition to reviewing potential failure modes, the 2009 PFMA update also reviewed and updated the "risk reduction actions" from the 2004 PFMA.

Based on the excerpts above, it is noteworthy that the potential for serious operational problems was identified even at "lesser flows." However, this potential failure mode was ruled out because it was judged to not represent a safety risk, since it was assumed only that vegetation would be uprooted and the potential for deep erosion was not recognized. It is also noteworthy that the cited emergency spillway capacity is more than thirty times higher than the emergency spillway peak discharge which occurred during the February 2017 incident (12,500 cfs).

## 5.0 2014 PFMA

In preparation for possible application of FERC's new Risk-Informed Decision-Making (RIDM) guidelines [F3-10], a new PFMA for the Oroville project was undertaken in 2014. For Oroville Dam and its appurtenances, the workshop duration was about two weeks, in contrast to the one-day workshops during the initial 2004 PMFA and the 2009 update. There were a total of 33 team members [F3-11]. The team included 20 "core team" members, including two consultant facilitators, four other external consultants who also completed the Ninth Part 12D review, an additional external consultant, ten representatives from DWR (including O&M headquarters, OFD, DOE, and DOE Project Geology), two representatives from DSOD, and a representative from FERC. The team also included thirteen "participants," including an external consultant, nine representatives from DWR (including O&M headquarters, OFD, DOE, DOE Project Geology, and others), two representatives from FERC, and a representative from Yuba County Water Agency. The DWR geologist who served on the core team had also represented DWR during the 2004 PFMA and the 2009 update.

The 2014 PFMA was clearly a much more extensive effort than the 2004 PFMA and 2009 update, in terms of investment of time and resources. As a result, in contrast to the limited PFMs identified in 2004 and 2009, during the 2014 PFMA, a total of 20 PFMs were identified and fully developed [F3-11], and an additional 31 "candidate" PFMs were identified but considered unlikely and ruled out (Category IV).

Three of the candidate PFMs (all ultimately placed in Category IV) are directly related to the actual events that occurred at Oroville Dam during the February 2017 incident, and are described below.

**Candidate F2** was identified as "PMF Event Occurring at Oroville Dam and Emergency Spillway is Overtopping and Head-cutting Occurs Initiating at the Feather River" and described as follows:

> "A PMF flood event is occurring and over 10 feet of water is spilling over the emergency spillway at Oroville Dam. Erosion begins where the flow is entering the Feather River and progresses by head-cutting into the reservoir."

Adverse and positive factors for this PFM were identified as follows, with the service spillway referred to as the FCO (Flood Control Outlet):

| Adverse Factors | Positive Factors |
|---|---|
| | Broad area below emergency spillway reduces energy of water. |
| | Have had 50 years of flow occurring at the downstream end of the FCO spillway channel, and very little erosion has been observed. A 150,000 cfs release through the FCO has also occurred. |
| | Distance from the Feather River to the emergency spillway is approximately ¾ mile. |
| | The rock between the Feather River and the emergency spillway is very competent and resistant to erosion. |

The rationale for not carrying this PFM forward was:

> "The failure of Oroville Dam through head-cutting was not considered credible because of distance from the emergency spillway to the Feather River (approximately ¾ mile), the competence of the rock, and the lack of erosion that has been observed at the end of the FCO spillway channel."

**Candidate F5** was identified as "Loss of Spillway Channel Lining Results in Erosion of Rock Underlying the Spillway" and described as follows:

> "Cavitation or slabjacking results in loss of the concrete lining in the spillway chute downstream of the FCO. The rock in the spillway chute erodes and the FCO is undermined and lost."

Adverse and positive factors for this PFM were identified as follows:

| Adverse Factors | Positive Factors |
|---|---|
| | The spillway channel concrete is in good condition and there is no evidence of significant erosion or stress resulting from flows experienced to date. |
| | The rock is fresh and hard and resistant to erosion. |
| | The duration of large flows through the FCO is not sufficient to develop significant erosion of the rock. |
| | DWR has performed minor repairs to the spillway concrete as recently as 2009. |

The rationale for not carrying this PFM forward was:

> "The spillway chute is in good condition and the underlying rock is very competent. Many spillways are constructed of rock with no concrete lining. It is seen as highly unlikely that the concrete lining will fail and highly unlikely that significant erosion of the rock will occur during one spilling event."

There is no mention of the 2013 spillway chute repairs in either the 2014 PFMA or the Ninth Part 12D report. Based on the above wording, it is likely that the repairs observed in the chute (actually performed within the prior year) were thought of by the participants as being from four years earlier. Thus, the durability of the repairs may have been misinterpreted.

**Candidate F6** was identified as "Scour of Soil and Debris During Flow over the Emergency Spillway Blocks the Feather River" and described as follows:

> "Flow over the emergency spillway during a large flood scours soil and trees from the slope as water flows over the emergency spillway to the Feather River. This blocks the river and causes the river to backup."

Adverse and positive factors for this PFM were identified as follows:

| Adverse Factors | Positive Factors |
|---|---|
| Creates adverse condition downstream. | The large flows would prevent damming of the river with debris. |
| | This would not result in a dam failure. |
| | The slope below the emergency spillway has relatively little vegetation and surficial cover and the underlying bedrock is not subject to significant erosion. |

The rationale for not carrying this PFM forward was:

> "Damming of the Feather River is seen as highly unlikely under the heavy flows that would be occurring if the emergency spillway is activated. Even if this did occur … there was no scenario that would result in failure of the dam or an uncontrolled release."

## 6.0 IFT EVALUATION OF 2014 PFMA

Although the potential failure modes relating to the service spillway chute slab failure and serious erosion downstream from the emergency spillway were essentially identified in the 2014 PFMA, the process fell short in three key areas when assessing these risks:

- It was incorrectly assumed that "very competent" rock conditions generally existed at the spillways, which would not allow for significant headcutting of the foundation back towards the crest structures.
- It was incorrectly judged that the spillway was in good condition; the significance of the repair history and its relationship to the vulnerabilities in the original design and construction was not recognized.
- Less importance was placed on these spillway failure modes, since they were believed to be very unlikely to result in catastrophic uncontrolled release of the reservoir.

The IFT interviewed many of the individuals involved with the 2014 PFMA to investigate the basis for these significant oversights in risk assessment.

### 6.1 PFMA Process Pressures

The first line of inquiry was in regard to pressures related to the 2014 PFMA process. Was it considered rushed, underfunded, understaffed, etc.?

From numerous interviews, the IFT heard a consistent story of a very thorough process with no serious budget or schedule issues. Two weeks of effort were spent on Oroville Dam alone, consuming the entire time that had been originally set aside for all seven dams within the FERC license. There was never a push to "wrap it up," the non-DWR participants all thought that DWR was very interested in undertaking the full process, and there was a strong motivation to "do it right."

Although the DWR participants were inexperienced in the PFMA process, and at first somewhat defensive and not open to the spirit of the process, non-DWR participants reportedly noticed a change after the first few days, and the full team became cooperative and engaged. One very senior, qualified, experienced and respected engineer opined that the 2014 Oroville PFMA was as thorough a PFMA as he had ever been involved with. By all accounts, this particular PFMA was very well done in terms of the FERC PFMA process, and there was no indication that the processes were "shorted" in any way during the workshop.

However, with the benefit of hindsight, some DWR personnel opined that they had not been given adequate preparation time to review the geological information in detail prior to the workshop, and that only a short review was undertaken to know what information was available.

## 6.2 Geological Misunderstandings

Regardless of the 2014 PFMA following the expected process and involving investment of considerable time and resources, the PFMA team fundamentally misunderstood the geological conditions pertaining to the spillway alignments.

The IFT believes this is partly due to the team relying heavily on statements by DWR's geologist, who was on the core team for all three PFMAs, had decades of experience with DWR, was highly respected, and had, in a 2005 memo, considered conditions directly underneath the emergency spillway weir and assumed the conditions to be similar further downstream, while not apparently recognizing that even the geologic data attached to the memo indicated significant depth of erodible rock (see Appendix C).

In the 2014 PFMA report section titled Major Findings and Understandings [F3-11], comments regarding geology are limited to:

> "10. Oroville Dam and related facilities (flood control outlet (service spillway), emergency spillway, intake structure, etc.) are essentially founded on competent bedrock. The bedrock is fresh and hard, jointed and fractured, with a few high angle structural shears and schistose zones in the dam foundation. The joints, fractures, shears and schistose zones are generally tight and discontinuous. Erosion of the bedrock and/or seepage in the bedrock is not expected to occur under confined conditions …"

Although a caveat is given in regard to "confined conditions," there is no mention of unconfined conditions such as downstream of the emergency spillway. Further, in discussing the PFMs related to uplift pressures, the following is noted under "Conditions Relevant to Flood Loading Potential Failure Modes":

> "3. Stability Analysis for FCO and Emergency Spillway Structures: … During the PFMA workshop it was noted that the FCO and emergency spillway structures are founded on highly competent amphibolite rock and the rock surface is rough ..."

This statement was corroborated in the PFMA report by construction photos of the foundations under these crest structures. However, no photos of the service spillway chute foundation were referenced. The report further stated:

> "4. Erodibility of Downstream Toe of Emergency Spillway: During the PFMA workshop, the potential for erosion of rock at the downstream toe of the emergency spillway weir was discussed. It was suggested that the amphibolite foundation rock is very competent, but may potentially erode by 'plucking' if subjected to a substantial jet of water. Upon further discussion, it was decided the ogee shape of the higher section of the emergency spillway weir will prevent a jet of water from occurring when spilling. Additionally, it was determined through drawing review that the concrete apron at the downstream toe of the ogee weir section is continuous over the length of the weir. The presence of the apron further reduces the potential for erosion at the downstream toe of the ogee weir."

For the emergency spillway it is further stated that:

> "If water were to spill over the Emergency Spillway, it would flow over natural terrain (shallow surficial deposits over bedrock)."

There is no discussion regarding the characteristics of the rock underlying the surficial deposits.

## 6.3 Elimination of Potential Failure Modes

During the 2014 PFMA, several potential failure modes related to the spillways were eliminated from further consideration due to incorrect geologic assumptions, incorrect assessments of the condition of the spillway chute, and/or a focus on considering only failure modes judged to potentially result in uncontrolled release of the reservoir.

Regarding Candidate F2, involving headcutting up the emergency spillway alignment, the IFT did not find any evidence that would have been available to the PFMA team that would have substantiated the claim of competent rock, except for the DWR 2005 and 2009 memos [F3-12 and F3-13] (see Appendix C). Also, the IFT notes that the following scenarios were not considered in the decision to not carry forward this failure mode:

- The possibility of headcutting initiating anywhere along the hillside, other than at the Feather River
- Use of the emergency spillway as an alternative backup to the service spillway in less than extreme flood situations, although this was considered in a separate PFM noted later in this section
- Failure of the emergency spillway or service spillway structures, which was excluded on the basis of the assumed good rock conditions and the assumed good condition of chute

Regarding Candidate F5, which involves erosion of the service spillway foundation, again, the IFT found no evidence supporting the claim regarding fresh and hard rock – this appears to be a transference of impressions of the adequately prepared rock foundations under the concrete gravity structures to the chute foundation, without reference to actual documentation. However, it was stated in interviews that participants remembered "pouring over" the foundation geology maps that were "rolled across the table." This is problematic, as these maps clearly show large areas of strongly weathered rock in the area of the initial chute failure on February 7, 2017. However, as reported in interviews, not all team members would necessarily have been around the table at the time these maps were reviewed. Also, as noted below in this section, it appears that the PFMA team did not directly access borehole logs, which contained the detailed rock descriptions. The ideas that strongly weathered rock could also occur at depth in portions of the hillside downstream of the spillway crest structures, and that strongly weathered rock could erode, were not developed.

There also appears to have been no review of any documentation that would support the opinions of the likelihood of chute lining failure – this apparently was based largely on the chute walkover during the PFMA process, relatively shortly after the 2013 repairs. The inspection report contains a single observation "The spillway chute is clear and the concrete is in good condition (Photo 18)." Interviews with various participants noted that there was no significant discussion regarding the concrete cracking or the underdrains, though DWR had been asked during the review whether there had been any turbid flows or sediment deposits. It is significant to note that neither the 2014 PFMA report nor the 2014 Supporting Technical Information Document (STID) include any service spillway chute drawings, either design or as-built. Further, during interviews, participants in the PFMA did not recall reviewing these drawings. Although there were two trigger mechanisms identified during the process of evaluating this potential spillway chute failure mode, cavitation damage and slab jacking, it does not seem that the discussion of the potential warning signs and trigger mechanisms was detailed enough for DWR staff to offer information related to the damage requiring repairs, the frequency of that damage, and the fact that flows significantly increase from the drains when the spillway flows. Someone familiar with slab jacking, its causes and warning signs, such as high drain flow, may have elevated the consideration of this failure mode if they were given this information.

The elimination of Candidate F6 exemplifies the current PFMA process emphasis on failure modes that would result in uncontrolled release of the reservoir. Clearly, the February 2017 incident shows that one does not require an actual failure of a water retaining structure to seriously affect the lives of thousands of residents, the environment, and the general organizational operations and goals of the owner. The PFMA process emphasis on uncontrolled release is also evident in the rationale for not carrying forward two of the Operational Candidate PFMs [F3-11]:

> "Candidate O1: Large precipitation event with power loss leads to activation of the emergency spillway
>
> Candidate O2: Gate at the FCO binds and limits opening of remaining gates leading to activation of the emergency spillway."

The rationale for not carrying either of these PFMs forward is:

> "It was determined that passing flow over the emergency spillway does not constitute an uncontrolled release, because it is functioning as designed. *While activation of the emergency spillway is not desirable, it is not considered a failure."* [italics added for emphasis]

Consideration of these potential failure modes does show that the use of the emergency spillway as a backup to the service spillway in non-extreme flood events was at least considered. However, from numerous interviews, the IFT formed the impression that most DWR staff and those involved in the PFMA studies considered the use of the emergency spillway in terms of only an "extreme" flood event. The IFT notes that a "1 in 100" year storm might be considered an "extreme" event in an operational sense, whereas, from a dam safety viewpoint an "extreme" dam safety would be a much larger storm, perhaps a storm with a return period of several thousand years or even approaching the probable maximum flood (PMF). Thus, different participants may have had different viewpoints concerning extreme events, which may have affected their evaluations of the likelihood of emergency spillway activation.

What should have been the expected likelihood the emergency spillway activation? Although the service spillway has a discharge capacity with all gates operating of about 300,000 cfs when the reservoir level approaches Elevation 917, serious flooding occurs at much lower discharges. As noted in Appendix C, spillway discharges into the Feather River exceed downstream levee channel capacity. The probability recurrence interval of this flood was given as approximately 450 years. The IFT was informed by DWR personnel that the rules contained in the 1970 USACE Flood Control Manual (governing Lake Oroville flood control operations) could result in activation of the emergency spillway while still controlling total releases from the service spillway to no more than 150,000 cfs. Thus, there should have been an expectation that the likelihood of emergency spillway activation in any one year would be in response to a flood with a return period of about 1 in a few hundred years. Had the probability of its use been considered to be more likely than only during the extreme flood event, there likely would have been more emphasis placed on the risks associated with potential erosion downstream of the emergency spillway crest structure.

Backwater from the spillway resulting in flooding of the powerplant was also considered under Candidate O6. However, it was rationalized that, although previous experience showed backwater conditions could necessitate the use of sandbags, the predicted rise in tailwater during the PMF would not flood the powerhouse, and, again, there would be no uncontrolled release of water.

As a further general indication of the reliance the 2014 PMFA team placed on the emergency spillway, it is noted in the report that:

> "The large emergency spillway capacity led to the ruling out or 'not-carrying forward' numerous PFMs related to extreme hydrologic loadings, operational error, seismic, and corrosion-related failures associated with the FCO and its radial gates. The emergency spillway would function as designed, routing flows and preventing overtopping of the dam and the saddle dams (Bidwell Bar Canyon and Parish Camp)."

## 6.4 Available Reference Documents

The IFT investigated the information available for the 2014 PFMA team to review, and whether the PFMA team had adequate access to information that would have assisted in properly identifying the actual quality of the rock at the spillways.

The IFT was told that more than 300 reference documents were made available to the PFMA team, and a list of 160 references is given in the 2014 PFMA report. A thumb-drive of technical background information had been distributed "at least four weeks prior to the workshop." The IFT assumes that this is the source of the 160 references.

The 2014 PFMA report notes the following statements, which mostly suggest a favorable assessment of the information available for review, though areas for improvement are also noted:

- "The thorough records that were kept during construction of Oroville Dam are very helpful. The ability to access this documentation made it possible to resolve some issues, which would have required exhaustive studies or explorations, had the documentation not been available. This made the Core Team and Participants aware of the need to develop and maintain good design and construction records for the current projects at the dam. The Final Construction Report has been particularly helpful. It is also important to have the design engineer's criteria for the project design, but the design criterion was not available for some questions asked during the PFMA."
- "The ability to access construction photos during the PFMA workshop is very useful. Consideration should be given to obtaining more historical photos for future use, if available, and organizing them by facility."
- "There is a wealth of information on Oroville Dam from past drawings, photos, and reports. In compiling the STID, the information included should provide value and minimize redundant information."
- "The STID concept is very useful for a project of this size and complexity. It is a living document in which resides key data and knowledge. It is important that the STID document be updated consciously and purposefully every 5 years with all the key information that is gained from recent studies, analyses, or inspections."

The 2014 PFMA report also notes the following comment, which further indicates a need for improvement in DWR's information management for its dams and their appurtenant structures:

> "The need for improving and updating data collection has become evident during the PFMA workshop …
>
> 2. Documents and data pertinent to the design, construction, maintenance, operations, and performance of the project facilities are distributed across several DWR Divisions and Offices. Recent efforts by DWR to consolidate and organize these documents to a central electronic repository or database should continue."

With regard to the quality of the rock at the spillways, among the 160 references listed in the 2014 PFMA report, one relevant reference is the Final Geology report for the Service Spillway [F3-14] (see Appendix C). However, none of the geologic references listed in the 2014 PFMA report would have included the detailed exploratory borehole logs from holes that were drilled along either the service spillway or emergency spillway alignments. In the case of the emergency spillway, it appears that no information at all pertaining to the geology downstream from the emergency spillway crest structure was accessed; this would have been available only in the Interim Geologic Data Report from 1962 [F3-15] (refer to Appendix C), which is not referenced. The IFT has also found that summary reports, such as the final construction report for the spillway, tended to gloss over the details that are contained in DSOD and Engineering Daily reports that identify problem areas during construction. Unfortunately, DWR could not locate a complete copy of the reference material given to the PFMA team to double check this likelihood of missing critical information that would have directly contradicted the geological statements given in the PFMA report.

The 2005 memo [F3-12] available in the STID, and which identified good rock conditions, referenced only boreholes in the foundation immediately underneath the emergency spillway weir. However even these boreholes showed significant depths (far in excess of 4 feet) of strongly weathered rock. The 2009 erodibility memo [F3-13] was also available for the 2014 PFMA, and it does show one –and only one – "typical" borehole log from the same area (see Appendix C). Although noted as being "typical," it was in fact, a log showing much less than typical depths of weathering. However, the 2009 study also accurately presents a table summarizing the borehole logs, which shows depths of strongly and/or moderately weathered rock up to 36.5 feet. Thus, the 2014 team members had access to information that could have pointed out that the assumption of good rock was probably erroneous.

There is also one note in the PFMA report that indicates that some thought was given to spillway drains, but the PFMA team was hampered by the lack of inspection reports:

> "53. The PFMA workshop would have benefited from prior inspection reports (or documentation of existing OFD inspections) of appurtenant structures. These include … FCO drains."

Again, FCO refers to the service spillway in the above excerpt. It is unclear whether this excerpt was in reference to the drains directly underneath the headworks structure, or those along the chute, as interviewees noted that there were specific presentations on the headworks stability and discussion of the headworks drains during the PFMA discussions. The fact that there was very little in the way of drain inspections or observations for the spillway chute (see Appendix F1) was not specifically noted.

## 6.5 <u>Discussions During the PFMA Workshop</u>

During the 2014 PFMA, although there was some discussion on the presence of shear zones and clay-filled joints, the DWR geologist reportedly expressed strong opinions and confidence in the quality of the rock underneath of and downstream of the spillways, and there was no disagreement within the team that the rock would not erode back to the reservoir. One interviewee remembered that the geologists did not say the rock would not erode, but “it was more like they said it won’t erode to failure.”

Why the DWR geologist was not questioned further is unclear, although the IFT was told that the team “pushed hard” on many other failure modes. In the case of the DWR geologist’s opinions, it was repeatedly emphasized to the IFT that the stated opinions were “more than strong,” and that the opinions and statements influenced the PFMA process.

### 6.6 Composition of the PFMA Team

The IFT was told in various interviews that the issues and PFMs that the Oroville Dam 2014 PFMA team focused on was in part a function of the individuals involved. There was ample time spent on the headworks, tendon failures, etc., as these aspects were of interest to the structural engineer(s) on the PFMA team, but there were no similar advocates for the spillway chute itself. Thus, the chute was not a priority, and there was significantly less time spent on review or discussion of the spillway chute design and construction, or the defects and repair history of the chute slab.

The IFT was also told that the number of DWR personnel was overwhelming to some of the external participants, and although the external consultants did not think that they had been overly influenced, the DWR personnel were over-represented in the PFM voting process. However, in numerous interviews with PFMA team members, reliance on the engineering judgment of others on the team was repeatedly cited – DWR participants felt they could and should place confidence in the external consultants, whereas the external consultants felt they were influenced by some DWR staff and relied heavily on some DWR statements (e.g. with regard to geologic conditions at the spillways).

One participant summed up the PMFA’s failure to identify the conditions that led to the February 2017 incident as a result *solely* due to factors related to the individuals comprising the PFMA team. However, as argued below, the IFT believes that process factors also played a role in the 2014 PFMA falling short in identifying the factors that lead to the February 2017 incident.

## 7.0 GENERAL COMMENTS ON THE PFMA PROCESS

Based on the IFT’s evaluation of the 2004, 2009, and 2014 PFMAs, as well as its evaluation of the PFMA process in general, the following general comments on the PFMA process are provided.

Some general comments on the PFMA process and the Oroville Dam PFMA were given in the 2014 PFMA report [F3-11]:

> “46. The PFMA process is time consuming, expensive, occasionally tedious, sometimes frustrating, but absolutely necessary and ultimately very thorough and very useful. The success reflects great credit to the facilitators in particular, but all participants in general.”

> “48. The process used in this PFMA was different from typical PFMA’s. In this PFMA, all PFM’s were listed in a ‘brainstorming’ session and then lists were revisited and screened to determine if the individual PFM’s considered should be

> carried forward and developed. The initial brainstorming is useful. However, the intermediate screening step may have added unnecessary additional time to the process. In future PFMA workshops, it may be better to do initial brainstorming and development of a 'long-list' of candidates, and then go directly to ruling out candidates, and selection and full development of potential failure modes to be carried forward as one step without the intermediate screening step."
>
> "50. The PFMA workshop for Oroville Dam required about two weeks to complete. Possibly more review time prior to the PFMA workshop would allow the PFMA to move more quickly."

These comments suggest that the 2014 PFMA team found the process to be relatively arduous, which is not surprising when considering that a large group was meeting all day, for numerous consecutive days, so there appears to have been a motivation to improve and streamline the process. These aspects were investigated by the IFT during the interviews, and are discussed below.

### 7.1 Definition of Failure and Failure Mode

It is evident that failure modes can be excluded from detailed consideration simply due the definition of "failure" which is used. Although "failures" such as the Oroville Dam service spillway chute failure are generally recognized, their importance and impact can be easily overlooked. From the Federal Guidelines for Dam Safety, Glossary of Terms [F3-16]:

> "Dam Failure. Catastrophic type of failure characterized by the sudden, rapid, and uncontrolled release of impounded water or the likelihood of such an uncontrolled release. It is recognized that there are lesser degrees of failure and that any malfunction or abnormality outside the design assumptions and parameters that adversely affect a dam's primary function of impounding water is properly considered a failure. These lesser degrees of failure can progressively lead to or heighten the risk of a catastrophic failure. They are, however, normally amenable to corrective action."

From numerous interviews, including interviews with FERC personnel, the IFT heard the opinion that the current PFMA process is focused on outright failure and uncontrolled release of the reservoir, not undesirable performance and public impact, even if these consequences are on a large scale. In current industry practice, there is little thought concerning what partial failure or component failure of a structure would actually mean. Being driven by the uncontrolled release of water, the focus is on water retaining structures, and the IFT believes a broader interpretation and definition of "failure" is needed.

Another important aspect is that the definition of "failure mode" as a chain of events has the potential to lead some practitioners of PFMAs to look for a single "root cause" leading down a relatively simple path to failure. This can result in a PFM being screened out of further consideration based on a single mischaracterized factor (such as rock quality). The IFT believes that more complex systems with multiple types of factors, including various human and operational aspects that can both initiate and interact over time with physical processes, must be considered. With these complex failure systems, any step in the process that is not well known could be tagged for further study. For example, how likely is it that cavitation or slab jacking will fail the chute slab?

### 7.2 Emphasis on Loss of Life

The dam safety community focuses efforts on protection of human life, and this is understandable. However, with this emphasis on loss of life consequences, failure modes that do not necessary involve loss of life as a consequence can be overlooked. For example, during the 2014 PFMA, a potential failure mode following an erosion pathway towards the right side of the emergency spillway near Monolith 3 was considered, but was not fully evaluated because life loss was not expected in the event of a breach at that location:

> "Breach outflows will be limited by 300 feet of rock upstream of Monolith 3 that is only 2 to 4 feet below the top of the monolith such that incremental life loss is not expected." [F3-11]

Regardless of the fact that lives may not have been lost, such an event and the subsequent flooding damage would likely not be considered tolerable by the downstream population. Also, the loss of one monolith could trigger a chain reaction, ultimately leading to the failure of several monoliths due to the concentrated flow at the base of adjacent monoliths. A responsible owner needs to have a full understanding of all consequences, not simply loss of life. Otherwise, the owner cannot make informed decisions on how to modify the dam, or what to do in the event of a failure of a spillway during a flood.

### 7.3 Emphasis on Dams over Appurtenant Structures

From numerous interviews, it was evident that, during PFMAs and other evaluations, attention to appurtenant structures such as spillways can be "eclipsed" by attention to main dams. From both DSOD and FERC personnel, the IFT heard that spillways are considered the "little brother" to dams with respect to engineering attention, and this is considered to be a pervasive attitude of most dam owners, and even among regulators. PFMA team participants tend to be selected based on their expertise related to the dam and possibly the spillway headworks.

### 7.4 Team Dynamics and Engineering Judgment

From numerous interviews, the IFT gained much insight into the dynamics of PFMA teams with respect to bringing expertise to bear, interactions among team members, and group judgment and decision-making.

One key theme which emerged is that the process is only as good as the technical understanding of potential failure modes the team brings to the process. The process relies on the strength of the individuals to know the PFMs prior to the workshop and then to apply that prior knowledge in order to "brainstorm" effectively, in an informal group setting and process which does not use explicit system models.

Another key theme is that various aspects of "group-think" can compromise the judgment and decision-making of the PFMA team, due to largely *subconscious* cognitive processes and group dynamics which result in team members aligning their views too readily in order to preserve group harmony, rather than engaging in sufficiently thorough, critical, and constructive discussion and debate [F3-20]. In the context of PFMAs, examples of potential group-think mechanisms include the following:

- Individuals perceived as experts by members of a group can highly influence the whole decision-making group when expressing strong opinions, and other members of the group may be reluctant to challenge such individuals, even if they believe they may have relevant information of which the rest of the group is not aware.
- Due to reputational and business considerations, individuals recognized as being experts may drift into confidently presenting opinions on topics outside their core expertise, rather than openly indicating the limitations of their expertise in a group setting.
- Individuals who speak first on a topic tend to have a disproportionately large influence on the opinions and decisions which the group ultimately reaches.
- Due to "diffusion of responsibility," individuals may rely on the effort and judgment of others in the team to evaluate and categorize failure modes, rather than forming and presenting their own opinions. Their subsequent lack of disagreement with the opinions expressed by others may be incorrectly interpreted as tacit agreement with those opinions, resulting in the group overestimating the level of group consensus.
- To the extent that an opinion is perceived to reflect consensus among the group, individuals tend to be reluctant to challenge such an opinion, and will instead tend to censor themselves.
- Individuals may be deferential to supervisors and take visual leads from them when deciding on failure mode categorization.
- Time and cost pressures motivate groups to expedite reaching consensus, rather than continuing discussion and debate.
- Group interactions tend to foster overconfidence in the validity of group opinions and decisions, such as related to categorization of PFMs.

Judging from interviewee comments regarding the Oroville Dam 2014 PFMA, all of these group-think mechanisms could have been in play to some extent. Again, it is noted that group-think mechanisms operate primarily on a subconscious level, so a group will typically not be aware that it has been influenced by group-think [F3-20], and that appears to be the case with the 2014 PFMA team.

## 7.5 Reliance on Available Documents

Any PFMA study must rely on available documents, and the IFT learned from interviews with both FERC personnel and external consultants that there are potential shortfalls in two of the most important of these; the STID, and the Part 12D Safety Inspection Reports. The IFT was told that often Part12D Inspection Boards mainly look at what happened since the last Part 12D Inspection; in the case of Oroville Dam, eight prior Boards had reviewed the Oroville Dam materials. Focus is placed on changes since the prior Board and how that Board's comments and recommendations have been addressed. If no recommendation is made to do more analysis or dig deeper into the records, there is no new information for the next team.

An STID is required in support of the Part 12D Safety Inspection Report. However, the preparation of a comprehensive document is a daunting, time-consuming task. FERC personnel noted that it is difficult to receive appropriate documents to cover all aspects of a project. The Oroville STID, for example, did not contain the relevant information required to fully and properly assess the spillway foundations and the spillway chute design and construction. It did not include the spillway foundation geology maps or the service spillway chute drawings. An effort needs to be made to improve the available records on each subsequent study, but without specific recommendations this may not happen.

### 7.6 Categorization of Failure Modes and Follow-Up

FERC categorizations of failure modes have been presented earlier in this appendix. According to a FERC Part 12D Refresher Training module [F3-17]:

> "If you do not fully develop a PFM, you cannot categorize it."

However, interviewees noted that, in reality, there is never enough time or information to follow up on and fully develop all PFMs, yet further input is often necessary to properly categorize them.

Due to the definitions of the various categories, it was noted in an interview that the actual failure modes in the 2017 spillway incident would have never been classified as Category 1, since there was no thought of uncontrolled reservoir release. Had they been a Category 2 or 3, the PFMA team would have given recommendations for further investigations. However, it is unlikely that such investigations would have been undertaken prior to the February 2017 incident, due to other higher priority issues within DWR.

In general, by emphasizing potential loss of life as a prioritization factor, it is likely that most dam owners would rank potential spillway issues quite low in an organization's risk inventory.

## 8.0 INITIAL REVIEW BY THE USACE

Shortly after the Oroville incidents, the US Army Corps of Engineers (USACE) Risk Management Center prepared a memorandum [F3-18] in order to have "… a quick look at the incident to ensure we haven't overlooked anything major in our processes."

As noted above, the USACE uses PFMA techniques as part of its risk analysis process. They have, however, developed their own internal methodologies. The USACE asked itself the question:

> "So, what would have been the likely outcome if USACE methodology had been applied to the Oroville Dam spillways prior to the incidents?"

The findings, while necessarily conjectural, echo a number of themes that have been explored in this appendix, and identify that the outcome would likely have been largely similar to that from the Oroville 2014 PFMA:

- "The two potential [spillway] failure modes … would likely have been identified."

- "The fact that the service spillway had experienced significant flows in the past without incident would probably be weighted heavily by the team in evaluating the probability of failure (although case histories suggest past good performance does not necessarily mean good future performance)."
- "… the estimated probability of reservoir release and associated risks would probably be low due to (1) the long erosion path, (2) competent rock at depth and under the spillway chute and control structure foundation …"
- "It is possible but unlikely that failure of the emergency spillway due to headcutting erosion and undermining would have made it to a 'risk driver' potential failure mode. This would be largely driven by the probability of a flood large enough to require discharges over the emergency spillway. It is possible, but unlikely that this potential failure mode would be linked to the service spillway potential failure mode in that discharges would be directed over the emergency spillway at smaller floods due to needed repairs to the service spillway damage."
- "So, it is unlikely that all the events that occurred at Oroville would have been envisioned by a USACE risk assessment team. The chance for significant damage would likely have been identified, but the chance of losing control of the reservoir (which never occurred at Oroville) would likely have been estimated to be small, as well as the incremental consequences of spillway control structure failure. Therefore, the risks would have also likely been estimated to be small …"

Preliminary conclusions given include:

> "6. It should be noted that risk assessments typically look at the potential for uncontrolled reservoir release, which means there could be significant damage and erosion requiring extensive repairs, but the chance for pool release and risk could still be small.
>
> 7. The one area where this can change how we look at risk is that typically event trees that end without breach of the reservoir are dead-end branches. Damage to Oroville's spillway [caused] significant changes to its operation. This could affect other failure modes, and this scenario is rarely considered."

Under "Future Considerations":

> "11. When Potential Failure Modes are evaluated during a risk analysis, events that can be caused by unplanned operation decisions, or due to poor or unexpected performance of another feature on the project, may not be captured."

As discussed further below, these comments from USACE highlight the need for considering interactions between system components when developing and evaluating potential failure modes. It also indicates that a strong focus of uncontrolled release of reservoir tends to dismiss this type of failure mode that can be significant in terms of impacts to the owner. While the federal government (USACE, FERC, and Reclamation) is seen as having deep pockets, and therefore, more focused on loss of life than infrastructure damage, most other dam owners may see the cost of repairing a damaged spillway as being very significant.

## 9.0 THE CASE FOR PROCESS REVIEW

One regulator noted during an interview that any PFMA is only as good as the input data, the time available, and people involved. And, quoting a retired FERC employee involved with the early development of the PFMA process:

> "Too many of the consultants who do PFMAs do not have 'extensive knowledge' (of failure modes). In my opinion, this is a very weak link in the chain."

The IFT recognizes that the qualifications to be a consultant are not extensive and detailed and could be improved to require more specific expertise. However, based on the IFT's findings as presented herein, it is argued that more needs to be questioned than simply the input, time available, or people involved. The process itself requires critical review. Quoting again from the retired FERC employee:

> "PFMAs were supposed to be an initial step towards risk assessment, a little step to get owners, consultants, and FERC staff thinking beyond standards. Unfortunately, FERC never moved on. After 15 years not much use was actually made of the process. Many owners saw it as 'checking' another FERC box …"

Although based on informal brainstorming and facilitated discussion, the 2014 PFMA team essentially identified the failure modes that actually occurred at the service and emergency spillways in February 2017. However, these potential failure modes were not thoroughly assessed due to fundamental misunderstanding of the geology at the spillways (see Appendix C), and possible a lack of understanding of the critical factors indicating that these failure modes could develop. There was also "normalization of deviance" with regard to the cracks in the service spillway chute, the short life of chute repairs, and the abnormally large drain flows during spill events (see Appendix F1), and the evidence indicates that the PFMA team did not adequately investigate these critical issues that had been observed from the time of first operation. In addition, it appears that the PFMA team did not review the service spillway chute design drawings, construction records, and performance and repair records.

The PFMA process did not identify the various inadequacies that led to the February 2017 spillway incident because the somewhat informal workshop setting was subject to groupthink effects, and the process required individuals to subjectively determine the relevance of the various failure modes, often relying on the expertise of others, without fully mapping the sequence from component-level failure to overall system level failure. The Oroville incidents were not the result of any particular root cause leading down a simple chain of events to failure. Rather, the incidents were the result of numerous disparate, relatively common physical, human and organizational factors interacting, over the course of decades, with the two primary physical causes of the incident; these being:

- The significant depth of strongly weathered, erodible bedrock
- Vulnerabilities included in the chute slab design and construction

Such "systems" thinking is difficult to capture through current PFMA practice, due to an emphasis on extreme events and total loss of water retention. Combined with an over-reliance on inherently fallible engineering judgment at both individual and group levels, this allows elimination of failure scenarios without enough factual information being adequately reviewed.

The following, from the preface from *Operational Safety of Dams and Reservoirs* (2016) [F3-19] is in many ways prescient of the Oroville incident:

> "The approach to the analysis and understanding of operational safety of dams and reservoirs developed in this volume starts from the observation that the dominant risks to be managed in dam safety derive not from unique events but from adverse combination of more usual events. One might think of these as *unusual combination of usual conditions* … These include the systems interactions among management policy, procedural factors, instrumentation and SCADA systems, operational and maintenance practices, design flaws, construction compromises, deterioration, outside disturbances, and many other things that are often overlooked at the time of design. The result is that many system failures do not fit within a traditional engineering-analytic framework and a new systems framework is needed."

Every one of the systems interactions listed in the above excerpt (except for SCADA) has been cited by the IFT as interacting and ultimately contributing to the Oroville Dam spillway incident.

## 10.0 POTENTIAL IMPROVEMENTS TO THE PFMA PROCESS

While it is beyond the scope of this investigation to lay out a full critique of the PFMA process and what should be done to improve it, the IFT did consider this aspect at a preliminary level.

Input in this regard was solicited during numerous interviews. Some interviewees seemed somewhat at a loss when asked what improvements could be undertaken, and FERC personnel opined that there is no easy answer, due to many conflicting objectives. When asked how to ensure that technical opinions are properly challenged, no interviewee had a ready answer. The IFT was told in a number of interviews that many owners simply see a PFMA as "checking another FERC box," completely missing the intent of getting the owners to better understand the risks and the consequences for which they are responsible, both ethically and legally.

However, numerous interviewees did have useful suggestions regarding how the PFMA process can be improved. Combining those suggestions with the IFT's evaluation of the Oroville Dam PFMAs and the PFMA process in general, the IFT identified numerous options that should be considered for improving the PFMA process:

- Provide more lead time to review documentation so that consultants and other participants can request additional information if documents provided are incomplete.
- "Possibly more review time prior to the PFMA workshop would allow the PFMA to move more quickly." [F3-11]
- "It may be advantageous to pre-prepare PFMs prior to the workshop." [F3-11]

- “The brainstorming sessions could be performed electronically prior to the workshop.” [F3-11]
- Provide the Independent Consultants with time to meet as a separate group, prior to the full workshop attended by all participants.
- Develop a “master list” of generic potential failure modes which serve as a starting point and checklist for developing project-specific PFMs, reflecting knowledge of failure modes which have actually occurred in the past - but do not allow the master list to limit the scenarios for PFMs which are developed and considered.
- To identify potential failure modes, combine brainstorming with a more structured approach during workshops.
- Ensure that the PFMA team is diverse and adequately covers all relevant technical disciplines with sufficient specialized expertise (e.g., although a structural engineer was present in the 2014 PFMA, there was apparently no engineer present who specialized in hydraulic structures).
- Divide the PFMA team into smaller, more specialized groups for different components of complex dam systems, but do not lose coordination on interactions between components. This would be an alternative to a single large group with different interests and specialties being together for a single PFMA workshop (e.g., one team for the dam, another for the spillways).
- Address the “burnout” factor associated with mental fatigue resulting from a long workshop process (e.g., break up the workshop into a series of separate workshops).
- Reconsider the definitions of the four FERC PFMA risk categories, and possibly increase the number of categories in order to make finer distinctions.
- If a voting process is used, consider including the qualifications needed to be able to cast a vote for particular PFMs, and possibly use silent voting so that participants are not influenced by the votes of others. Alternative, and in the IFT’s opinion preferably, assign PFMs to categories based on consensus, with outlying opinions recorded appropriately if full consensus is not achieved.
- Assure that the STID adequately covers all system components, and includes a history of repairs and their performance for all system components.
- Broaden the scope of failure modes considered in the PFMA process beyond those which necessarily result in uncontrolled release of water, but which may still have serious consequences for the dam owner, the public, other stakeholders, and the environment.
- Supplement the PFMA by way of an FMEA or similar process for complex structures (e.g. gated spillways).
- Assure that appurtenant structures get appropriate attention, commensurate with their significance.

Even if the above suggestions were implemented, budgetary and organizational constraints may prevent the current PFMA (and Part 12D) process from fully undertaking comprehensive evaluations of older infrastructure in light of current best practices for design and construction. For larger, more complex structures, this could take months, if not a year or more, by a dedicated team of competent professionals working from first principles and original data, rather than relying on past reports. The results would essentially be a relatively comprehensive, much expanded STID document that could then be used as the basis for reliable and quantified input to a risk assessment process. Such a substantial effort could be expected to remain valid for much longer than the current 5-year expectation if it is sufficiently comprehensive.

It is noteworthy that the Reclamation SEED process, described above in Section 1 of this appendix, is intended to provide this type of comprehensive review of available information and a “fresh look” to evaluate a dam and its appurtenances relative to contemporary state of practice, engineering theories, analytical methods, and understanding of failure modes. Indeed, the IFT believes that diligent application of the SEED process to the Oroville project would likely have identified the erodible foundation conditions at the spillway foundations, and would have also identified the spillway failure modes which occurred in February 2017.

Beyond the necessity of relatively comprehensive design reviews, in view of the evidence and arguments presented in this appendix, the IFT is led to the conclusion that a critical review of the current PFMA process itself is required. This review should not only look at the specific shortcomings of the current processes (as were identified in IFT interviews), but must also assess why the process allowed for elimination of failure modes that were, in the case of Oroville, fully plausible. The IFT suggests the strengths and weaknesses of the PFMA process be compared against risk assessment processes in other industries, including FMEA (as discussed above), various analyses based on systems theory, and fault tree analysis. Some of these methods first develop a detailed understanding of how a system performs properly, and only then is consideration given to how component failure can lead to overall system failure.

Although a very useful tool, and likely quite adequate for a majority of smaller dams, it must be recognized that the current PFMA process can have difficulties in properly characterizing risks for large, complex systems, including accounting for human and operational aspects in failures. In addition, the current PFMA process does not explicitly consider how broader organizational factors, such as culture and decision-making authority and practices, can contribute to failure.

## 11.0 SUMMARY

For each FERC project, in conjunction with the regularly scheduled Part 12D 5-year inspections, it is required that a Potential Failure Mode Analysis (PFMA) be developed and updated, as needed. For Oroville Dam, an initial PFMA was developed in 2004, it was updated in 2009, and a new PFMA was developed in 2014.

The FERC PFMA is a multi-step process which involves assembly of a team, collection and review of available background information, site review and inspection, facilitated workshop sessions which involve open-minded team “brainstorming” to identify and evaluate potential failure modes

(PFMs) and their potential consequences, identification of risk management measures for potential failure modes, and documentation of the PFMA.

The FERC PFMA process is less structured and is fundamentally different than techniques practiced in other industries, such as Failure Modes and Effects Analysis (FMEA). The FERC PFMA has also not typically provided the level of comprehensive safety review espoused in Reclamation's Safety Evaluation of Existing Dams (SEED) Manual. The goal of the PFMA process is to identify all potential failure modes, and the dam safety community places great confidence in the process, with the expectation that it will provide a strong basis to help ensure the safe retention and passage of reservoir water. However, the process tends to eliminate PFMs that do not result in uncontrolled release of the reservoir, and by defining failure modes as a linear chain of events, tends to oversimply complex PFMs involving multiple interactions of system components.

In the case of Oroville Dam, the 2004 PFMA and 2009 PFMA update were limited efforts, in which the workshops were only one day each. A single PFM was fully developed in 2004, and an additional PFM was fully developed in 2009, but these PFMs were ruled out, and neither was related to the spillways. In both 2004 and 2009, the rock at both spillways was incorrectly described by DWR's geologist as being generally non-erodible, which is how the rock was described in DWR memos issued in 2005 and 2009, and other team members accepted this incorrect characterization without proof of its accuracy.

The 2014 PFMA for Oroville Dam was a much more thorough effort, with the workshop extending for about two consecutive weeks, and it was considered by some to be the most thorough PFMA completed for any dam in the US to that point. A total of 20 PFMs were identified and fully developed, and an additional 31 "candidate" PFMs were identified but considered very unlikely and ruled out. Three of the candidate PFMs which were ruled out are directly related to the events that occurred at Oroville Dam during the February 2017 incident:

- "Candidate F2" was identified as "PMF Event Occurring at Oroville Dam and Emergency Spillway is Overtopping and Head-cutting Occurs Initiating at the Feather River," and further described as "a PMF flood event is occurring and over 10 feet of water is spilling over the emergency spillway at Oroville Dam. Erosion begins where the flow is entering the Feather River and progresses by head-cutting into the reservoir." This PFM was ruled out on the basis that "the failure of Oroville Dam through head-cutting was not considered credible because of distance from the emergency spillway to the Feather River (approximately ¾ mile), the competence of the rock, and the lack of erosion that has been observed at the end of the FCO spillway channel."
- "Candidate F5" was identified as "Loss of Spillway Channel Lining Results in Erosion of Rock Underlying the Spillway," and further described as "cavitation or slabjacking results in loss of the concrete lining in the spillway chute downstream of the FCO. The rock in the spillway chute erodes and the FCO is undermined and lost." This PFM was ruled out on the basis that "the spillway chute is in good condition and the underlying rock is very competent. Many spillways are constructed of rock with no concrete lining. It is seen as

highly unlikely that the concrete lining will fail and highly unlikely that significant erosion of the rock will occur during one spilling event."

- **"**Candidate F6" was identified as "Scour of Soil and Debris During Flow over the Emergency Spillway Blocks the Feather River," and further described as "flow over the emergency spillway during a large flood scours soil and trees from the slope as water flows over the emergency spillway to the Feather River. This blocks the river and causes the river to backup." This PFM was ruled out on the basis that "damming of the Feather River is seen as highly unlikely under the heavy flows that would be occurring if the emergency spillway is activated. Even if this did occur … there was no scenario that would result in failure of the dam or an uncontrolled release."

Although these three candidate PFMs come close to capturing the spillway failure modes which actually initiated during the February 2017 incident, the process fell short in three key areas when assessing the associated risks:

- It was incorrectly concluded that "competent" or "very competent" rock conditions generally existed at the spillways, which would not allow significant headcutting back towards the spillway crest structures.
- It did not identify other signs that made the hydraulic jacking of the chute slab more likely (marginal design details, cracks over drains, frequent repairs to cracks and spalls, and large flows from the drains during spillway flows).
- Less importance was placed on these spillway failure modes, since they were believed to be very unlikely to result in catastrophic, uncontrolled release of the reservoir and loss of life.

The same geologist represented DWR during all three PFMAs, and this individual had decades of experience with DWR, was highly respected, and was the author of the 2005 memo in which geologic conditions at the emergency spillway were mischaracterized. Based on numerous interviews, the IFT believes that the PFMA team heavily relied upon and deferred to the opinion of this DWR geologist. However, there were also other shortcomings in the process which contributed to the 2014 PFMA team mischaracterizing the geology at the spillways and not recognizing the substantial depth of erodible material:

- More than 300 documents were reportedly made available to the team, of which 160 were referenced in the PFMA report. It appears that, among these 160 references, one relevant reference was a final geology report for the service spillway, however none of the references included the detailed exploratory borehole logs from holes that were drilled along either the service spillway or emergency spillway alignments and documented in preliminary and interim geology reports.
- During the workshop, some team members were "pouring over the foundation geology maps that were rolled across the table." These maps clearly showed large areas of strongly weathered rock in the area of the initial chute failure on February 7, 2017, and the erodibility of this material was apparently not recognized.

- DWR's 2005 and 2009 memos for the emergency spillway were available to the 2014 team. While the narratives in both memos incorrectly characterized the hillside downstream of the emergency spillway weir as consisting of non-erodible rock below a few feet of soil, both memos contained data which clearly indicated that the depth of erodible material was up to tens of feet in numerous locations, and this was apparently not recognized.

More generally, from numerous interviews and the IFT's review of the PFMA process, the IFT identified a number of key limitations of the process:

- The process is focused on large-scale failure and uncontrolled release of the reservoir, with limited consideration given to partial or operational failure modes, which can still have major consequences for the dam owner, the public, other stakeholders, and the environment. Reflecting industry norms, the process also gives less consideration to appurtenant structures of dams, such as spillways. The IFT believes that a broader interpretation and definition of "failure" is needed, which a) accounts for partial and operational failure modes associated with all system components including dam appurtenances, and b) includes consideration of failure modes which account for complex interactions among physical and human system components, rather than only failure modes involving simple linear chains of events.
- The effectiveness of the process is dependent upon the technical understanding of potential failure modes which the team brings into the process. For example, for the 2014 PFMA for Oroville Dam, by combining all features into one meeting (in fact, the PFMA originally was intended to cover multiple dams), discussion specific to hydraulic spillway failure modes appears to have been a weak link, and for most of the team, the chute and unlined emergency spillway area seemed to be of little interest.
- Various aspects of "groupthink" can compromise the judgment and decision-making of the PFMA team, due to largely subconscious group dynamics which result in team members aligning their views too readily in order to preserve group harmony, rather than engaging in sufficiently thorough, critical, and constructive discussion and debate. Judging from interviewee comments, there may have been a degree of group-think during the 2014 PFMA.
- The process requires individuals to subjectively determine the relevance of the various failure modes, often relying on the expertise of others. There is an over-reliance on inherently fallible engineering judgment at both individual and group levels, which allows elimination of failure scenarios without enough factual information being adequately reviewed.

Although a very useful tool, and likely quite adequate for a majority of smaller dams, it must be recognized that the current PFMA process can have difficulties in properly characterizing risks for large, complex systems, including accounting for human and operational aspects in failures. In addition, the current PFMA process does not explicitly consider how broader organizational factors, such as culture and decision-making authority and practices, can contribute to failure. Weaknesses in the PFMA process have also been recently identified by the USACE following the

Oroville incident, and they opined that all the events that occurred at Oroville would have been envisioned by their PFMA process.

Drawing on comments from interviewees, the IFT identified numerous options that should be considered for improving the PFMA process. However, even if these options were implemented, there typically will not be enough time in the current PFMA (and Part 12D) process to undertake relatively comprehensive design reviews, as described in the Reclamation SEED Manual. The IFT believes that such comprehensive reviews should periodically be conducted.

Beyond the necessity of periodic relatively comprehensive reviews, in view of the evidence and arguments presented in this appendix, the IFT believes that a critical review of the current PFMA process itself is warranted. Such a review should compare the strengths and weaknesses of the PFMA process with risk assessment processes used in other industries and by other federal agencies.

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# Appendix G

# Spillway Chute Repairs

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## 1.0 GENERAL

This appendix covers the repairs of concrete in the service spillway chute only. Repairs to the chute walls, headworks, and terminal structure are not covered. The general description of the service spillway (Flood Control Outlet or FCO) provided in Appendix A.

## 2.0 SPILLWAY CHUTE DAMAGE

Much of the observed spillway chute damage that has required repairs was caused by various different mechanisms: cracking, removal of joint filler, and delamination/spalling. While hairline cracking due to the initial temperature drop following curing and concrete shrinkage can be expected in concrete chute slab placements, the observed cracking in the Oroville service spillway chute predominantly occurs over the herringbone drains, and were first documented in a 1969 report [G-1]. This report indicates that cracks appeared above herringbone drains during curing, within one month of placement. The report indicates that only two of the cracks had “significant opening,” the remaining cracks were not described as “hairline,” a descriptor used elsewhere for other cracks, so the IFT assumes that the openings were more than “hairline,” but at the time, less than “significant opening.” The report concluded that “the herringbone drain lines reduce the effective invert slab section apparently more than do the ‘weakened plane’ contraction joints.”

Although these cracks appeared very early during construction, there apparently was no design change during construction that would have minimized the cracking potential. Seasonal temperature changes result in concrete cooling and contraction during the winter months. This cooling causes these cracks to open as if they are contraction joints, allowing any water flowing in the chute to enter the foundation beneath the chute. During the IFT site inspection in April 2017, virtually all the herringbone drains in the remaining upper chute were observed to have some vertical cracking above them. These cracks had been repaired since the February 2017 incident, so the IFT could not gage the crack openings at that time.

The second problem is associated with opening of the formed contraction joints and removal of the ¼-inch by 1-inch filler material during spillway flows. With a winter flood season, spillway flows typically occur when the chute concrete is at its coolest state. The contraction of the cool concrete results in contraction joint opening, which allows the filler material to be more easily removed by flowing water. This filler material was installed to create a water tight barrier at the joints. Although all the contraction joints were to be constructed with shear keys, opening of the joints provides a flow path for water in the chute to enter the foundation.

The third problem is delamination and spalling at joints and cracks. Sand and sediment can accumulate in the open joint or cracks. This material causes problems in the summer months when the concrete expands in the heat. The non-compressible sediment that replaces the missing compressible joint filler causes compressive forces to be transferred to adjacent slabs as the gap closes. This action leads to spalling of the slabs near the joints. This same mechanism also leads to spalling at vertical crack locations associated with the herringbone drains. The spalling exposes rebar on the chute surface and dowel bars at the joints to corrosion, leading to rebar failure when subjected to tension during the winter contraction cycle.

While spalling at joints seems to be a common problem in spillway chutes, especially those that are south-facing, or experience freezing of water that collects in the joints in winter (freezing may not have been a significant problem at Oroville because of limited occurrence of freezing temperatures), spalling at cracks may be somewhat unique to Oroville. Typically, drains are placed below the chute slab, not within the slab where they can induce cracking (as at Oroville). Filler material is also rarely used on chute chute slab joints, because it is easily removed by flowing water. Many modern spillways constructed after Oroville have control joints where bonded rebar (as opposed to unbonded dowels) keep joint openings to a minimum. Another now common design feature is the use of waterstops at joints in the spillway chute, to prevent flow into the foundation.

## 3.0 REPAIRS

DWR provided the IFT with information relative to five major repair efforts in the spillway chute. Minor repairs may have also been performed over the life of the spillway, but details of these repairs were not provided. The documented repair efforts occurred in 1977, 1985, 1997, 2009, and 2013. These efforts will be summarized in this appendix. It is noted that these repair efforts occurred at intervals of close to a decade, except that the intervals was only 4 years between 2009 and the most recent 2013 repairs. From discussions and interviews with DWR personnel, it appears that no concerted effort was made to investigate the underlying factors causing failures involving cracking, delamination, and spalling and the subsequent need for repeated repairs including recurrences at the same locations.

### 3.1 1977 Repairs

The first major spillway chute repairs were completed in 1977, under Specifications No. 77-41. Drawings in these specifications included a table locating 66 required repairs. This table indicated that, of all of the repairs completed in 1977, 10 repairs were required along the contraction joint at Sta. 33+00. Repairs were to be made in accordance with Reclamation's Concrete Manual.

It appears that in 1977 repairs were made only to the contraction joints and spalls adjacent to the contraction joints. There is no reference to repair of cracks over the herringbone drains. According to the drawings, contraction joints were to be repaired by making sawcuts 7.5 inches deep and ⅜-inch wide, cutting through the dowels. Expansion joint material was to be placed in the sawcut joint. The contractor was also to make saw cuts 2 inches deep around the damaged areas and then remove concrete by chipping. Concrete was then replaced in these spalled areas. It is not clear if the sawcutting of the contraction joints were general repairs (applied to the entire contraction joint), or if they were only to be associated with areas where spalling concrete was repaired. The specifications paragraphs also refer to the contraction joints in the chute slab, along with the wall expansion joints, as existing expansion joints, but they are shown as contraction joints on the repair drawing. The chute slab joints were called contraction joints on the original design drawings, and had expansion material placed in only the top one inch of each joint. Designers in 1977 may have believed that the spalling at these joints was due to inadequate room for expansion, and, therefore, wanted them to be converted to expansion joints, but no documentation related to these designs and the construction was available for review by the IFT.

### 3.2 1985 Repairs

The only documentation of 1985 spillway repairs provided to the IFT was a memorandum dated September 23, 1985 [G-2]. This document includes photos of crack repairs, presumably above herringbone drains, and joint repairs. Photo captions indicate repairs were made at approximate Stations 29+00, 31+50, 33+00, and 34+50. It appears that the repairs included saw cutting on either side of a crack, or joint spall, chipping concrete to below the rebar, and rebuilding the area with new concrete.

### 3.3 1997 Repairs

Specifications No. 97-22 were prepared for the 1997 spillway repair work. The work description included sawcutting, core drilling and grouting, air/sand blasting, removal of concrete, concrete work, sealing cracks, and joint repair. The work was to be directed in the field, and the drawings did not show any details of the repairs.

A chute inspection report [G-3] describes the "location and description of concrete repair," which lists over one hundred areas of damage requiring repair between Station 13+00 and 36+50. These repair areas included 18 areas between Stations 33+00 and 35+00. Repairs to both lateral and longitudinal joints and lateral cracks were needed in this area. Between Station 33+45 and 34+65, lateral cracks at 20-foot spacing were described as needing repair "across all lanes."

Damage areas included failed previous repairs across lateral cracks and joints, presumably from the 1977 and 1985 repairs. However, there were more areas described as damaged repairs along cracks than there were identified as being repaired in the previous documentation. This would suggest that some undocumented repairs were made at earlier dates, or that the 1985 crack repairs were not fully described in the inspection report from that time period.

The chronology in the final construction report for the 1997 repairs [G-4] lists repairs in the vicinity of Sta. 34+00 being started on October 24, 1997. Photos included in this report were uncaptioned and were too grainy to show significant detail.

### 3.4 2009 Repairs

The 2009 repairs were well documented. Specifications No. 09-14 were prepared for these repairs. Documentation also included Field Notes consisting of drawing mark-ups (Figure G-1), a Final Construction Report [G-5], and numerous photographs of the work. The specifications drawings showed areas that were to be repaired by the contractor. The field note mark-ups show the areas that were actually repaired. Most of the construction photos were not captioned, so it is difficult to determine the location of the work shown in the photos. The repair work was completed by a private contractor under contract to DWR.

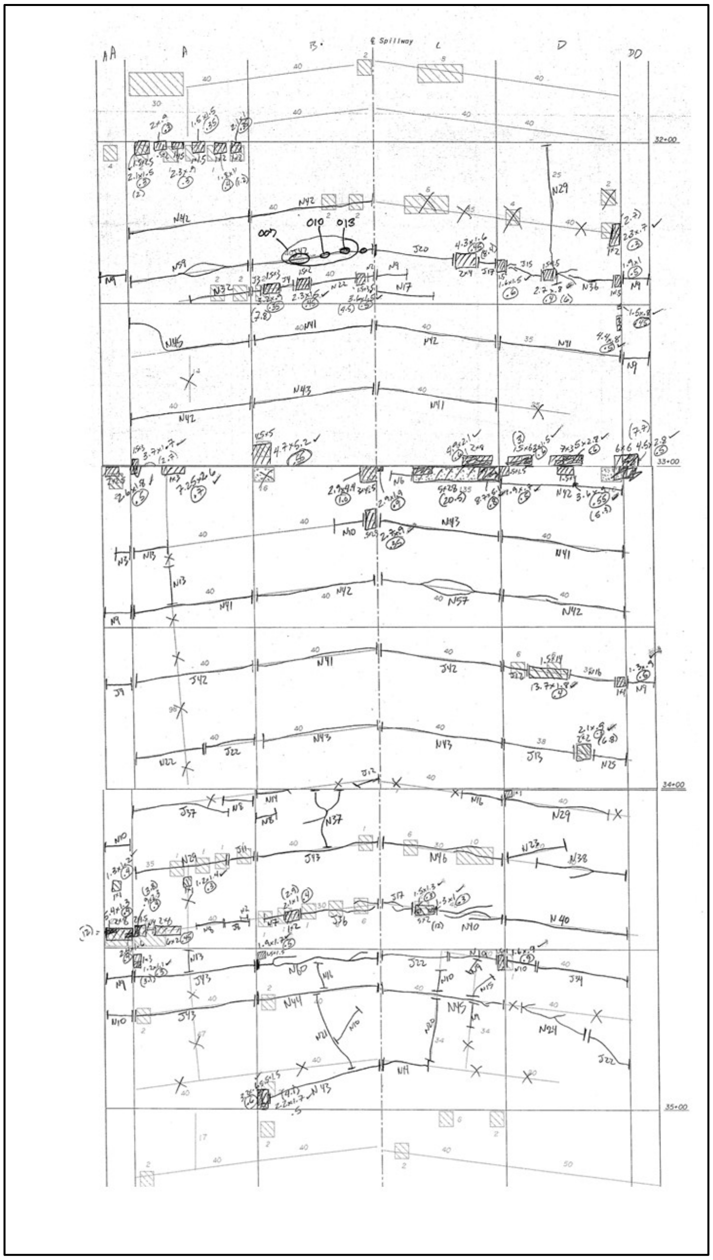


**Figure G-1*: Repairs made in 2009 (from Field Notes)***

The repair effort for cracks ranged from minor crack repair (1/4-inch wide or less) to major crack repair. Where spalls and/or delaminations (labled as "voids" on the drawings) were adjacent to cracks, the repairs were more extensive. Minor repairs called for the cracks to be completely filled with a low-viscosity epoxy. The epoxy used was a Sikadur 55 SLV formulation that is specifically intended to penetrate and seal cracks, driven by gravity only. While it worked well, most cracks required three to four applications to completely fill the volume of the crack, with a few localized spots requiring six to seven applications. Major crack repair included preparing the cracks by sawcutting or routing to a neat rectangular cross-section with minimum dimensions of 3/8" wide x 1/2" deep. The contractor used hand-held grinders to cut a rectangular groove at the top of these cracks. The design called for the major cracks to be completely filled with non-shrink grout. However, this method did not seem to work as planned because the cracks were acting as expansion joints. The contractor was required to grind the cracks deeper, place a foam backer rod in the crack, followed by Sikaflex elastomeric sealant. This revised repair is like the detail shown in the drawings for repair of the existing 3/4” expansion joints. At the time, there was some concern about durability of the elastomeric seal under spillway flows. Approximately one-third of the major cracks were repaired with non-shrink grout, and the remainder with elastomeric sealant. The grouted cracks are reportedly located right of centerline (looking downstream) from station 14+00 to station 36+00.

Spall repair areas, which were mostly near herringbone cracks or joints, were sawcut to a depth of 1-inch to 1-1/2-inch, and then jackhammered out, while leaving existing reinforcement intact. Reinforcement was found to be corroded 2 to 4 inches into the concrete and bars had failed in several locations. The rebar was cleaned and severed bars were repaired using Dayton Superior Bar-Lock, set screw type couplers. Immediately prior to replacing the concrete, SBR latex bonding agent was applied to the existing concrete and thoroughly worked into the surface with a brush, per the manufacturer’s instructions. However, the contractor applied the latex bonding agent without pre-mixing it with cement, as required by the manufacturer. This action could have resulted in reduced effectiveness of the bonding agent.

Some of the repairs as described above were the result of modifications made to the repair methods during the construction. These included a reduction to the area of concrete to be removed around a spalled area above the drains, a 1-1/2-inch sawcut above the crack in the new concrete, and splicing of broken rebar over the drains (see Figures G-2, G-3, and G-4). The modification to reduce the area removed around a spall (Figure G-2) was made to minimize the potential to damage the VCP pipe below as the concrete was being chipped out. This process seemed to inadvertently fracture the pipe when large concrete aggregates were present, and it was believed that by reducing the area of conctere to be excavated, the damage to the pipe would also be reduced. However, this change may have resulted in some cracked or delaminated concrete not being fully removed.

Sawcuts above the cracks in the new concrete (Figure G-3) were done because the DOE staff believed that the cracks had become “expansion joints.” The IFT believes that the cracks had opened as a result of concrete contraction, and the location above the herringbone drains corresponds to the reduced concrete thickness (from a minimum of 15 inches to 7 inches) above the drains. If the cracks below the new concrete were still open, a partial 1-1/2-inch sawcut above the crack in the new concrete would not prevent transfer of load as the concrete expanded in the summer heat. The thin section of new concrete between the sawcut and the cracked original concrete below the repair would be subjected to high compressive stresses, which could cause delamination of the repair. Rebar passing through the repair, whether spliced, or original intact bars, would go into compression during high temperature periods, possibly buckling the bars and contributing to the potential for concrete delamination and spalling. In the IFT’s opinion, these repairs were mearly temporary and cosmetic, and would not be capable of resisting the temperature stresses that caused the damage in the first place. The way in which they were done may have even potentially resulted in larger damaged areas in the future.

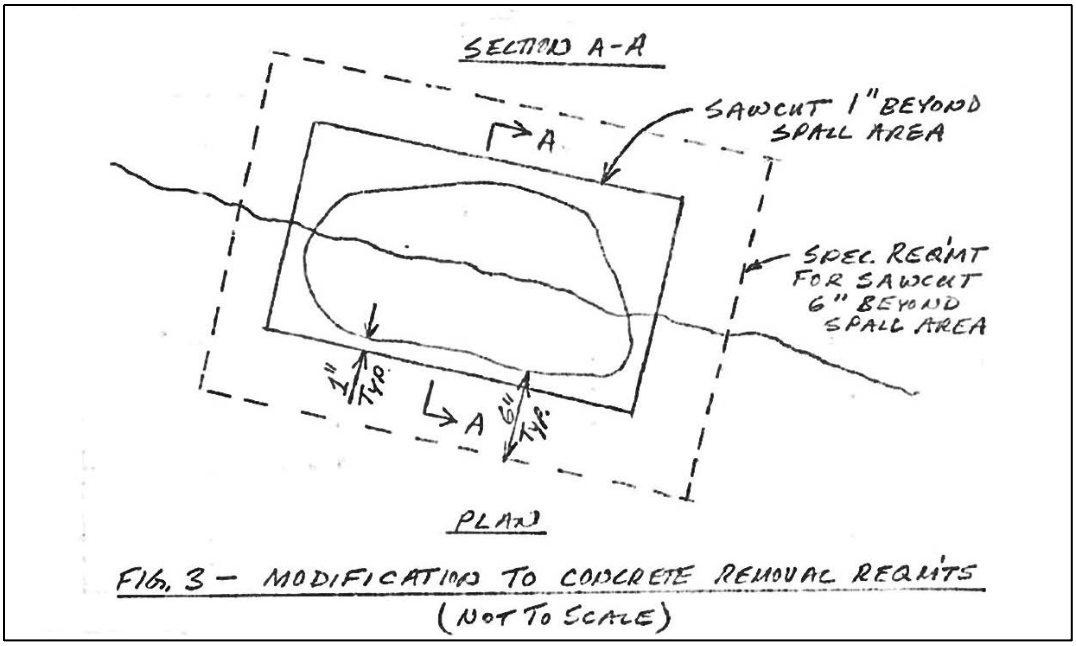


**Figure G-2: Revision to 2009 Repairs to Reduce the Area of Concrete Removal Around Spalls**

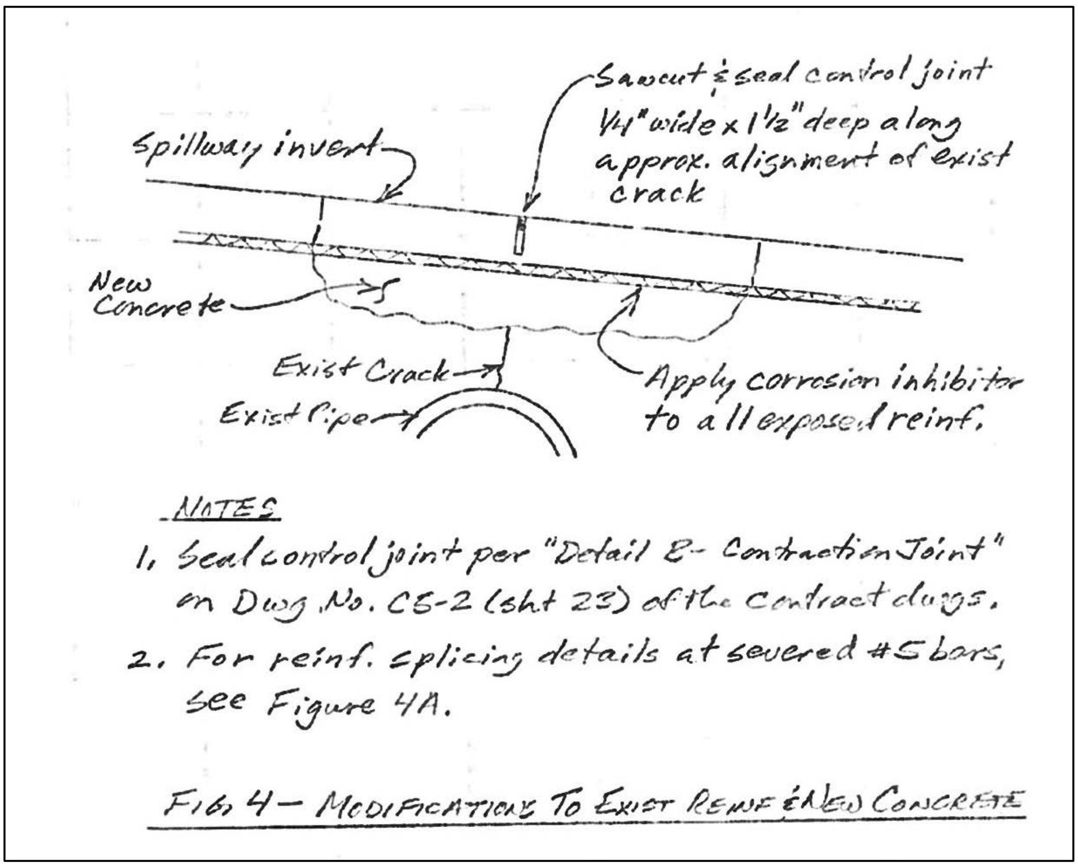


**Figure G-3: Revision to 2009 Repairs to Provide Sawcuts Above Cracks Over Drains**

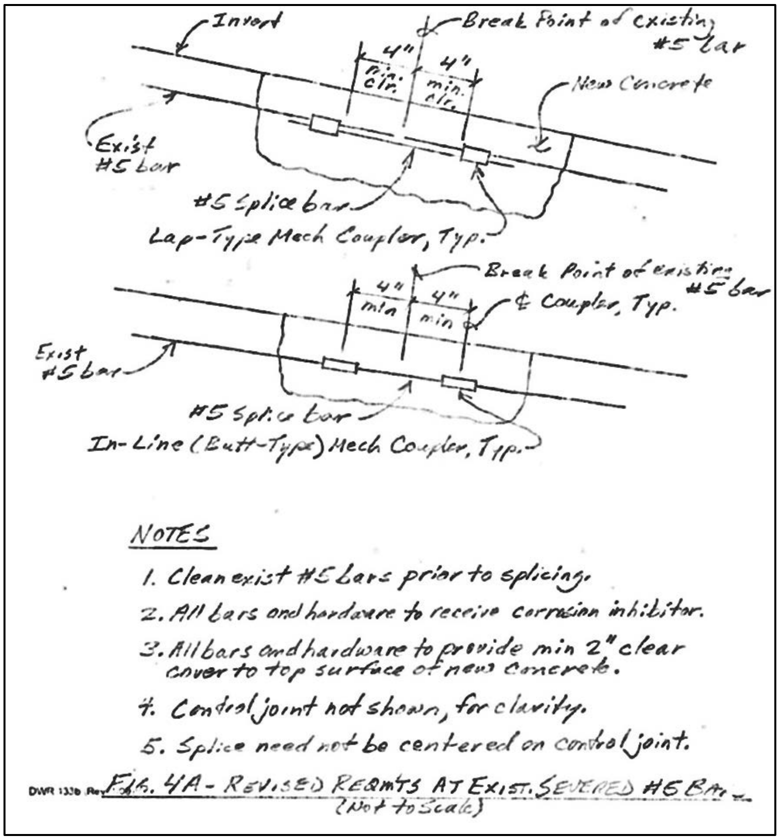


**Figure G-4: Revision to 2009 Repairs to Splice Broken Rebar Over Drains**

## 3.5 2013 Repairs

The 2013 repairs were made based on a September 17, 2012 inspection of the spillway [G-6]. It was noted that the chute slab was partially covered with algae, but in the areas that were exposed, it appeared that some of the 2009 repairs had failed (see Figures G-5 through G-7). There were places where the elastomeric sealant had failed at cracks over herringbone drains. This was thought to be due to the backer rods being installed too high, with too thin of a sealant cover. There were also spalls of some repair patches. An office memo [G-7] also documented the condition of the spillway and resulted in the following conclusions:

1. "The spillway repairs are holding up well although areas are beginning to degrade and fail with use.
2. The spalling and cracking of the concrete panels continues to occur.
3. The polyurethane sealant is performing much better than grout sealing. Most of the grouted cracks had washed out."

The memo recommended that repairs be made by OFD civil maintenance, followed by reinspection. Repair methods generally followed the methods specified for the 2009 repairs. The 2013 repairs are shown in Field Notes (Figure G-8).

**Figure G-5: Sealant Failure Over Herringbone Crack**

**Figure G-6: New Concrete Deterioration**

**Figure G-7: Spalled Concrete in Previous Repair**

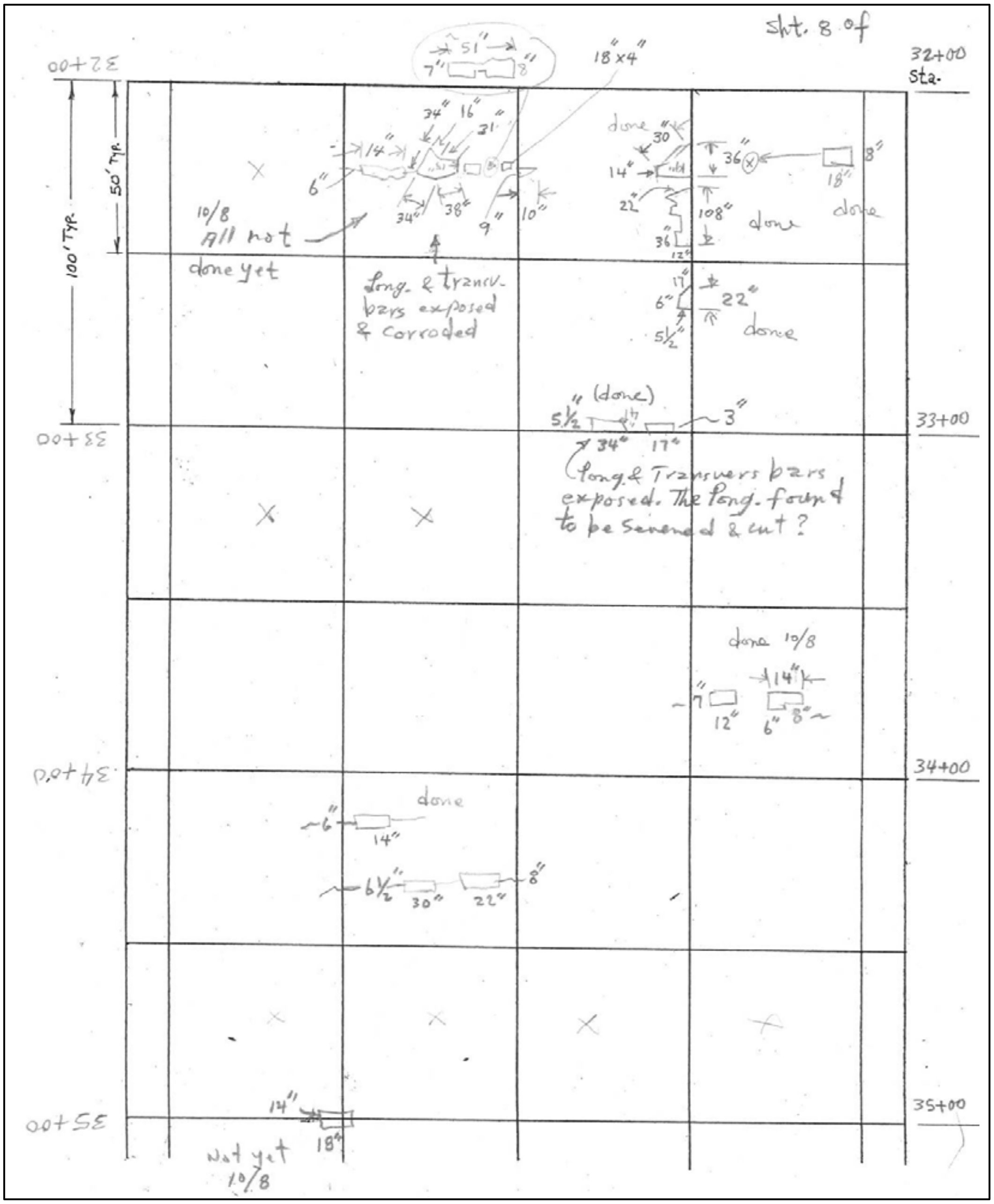


**Figure G-8: 2013 Repair Field Notes, Sta. 32+00 to 35+00**

## 4.0 IFT COMMENTS

1. Cracks began to appear over herringbone drains shortly after the initial spillway construction. These drains are spaced more closely than the planned lateral contraction joints. It is likely that the contraction joints at the 50-foot stationing that were prepared with a crack initiator on the slab surface (as opposed to being formed the full depth of the slab) did not develop to the full depth of the chute slab, because there was adequate stress relief at the herringbone cracks. The cracks over the herringbone drains and the joints at 50-foot stationing are crossed by the full rebar mat, and were not doweled like the

contraction joints at 100-foot stationing. Spalling at the crack locations may have been due to the rebar mat that was exposed to corrosive conditions as the cracks opened and allowed the unprotected bars to be exposed to water and air. Many of the bars that failed at crack locations also showed signs of corrosion.

2. Filler material in slab chute slab joints that relies on compression or weak material bond is not an appropriate repair for a spillway with high velocity flows. Repairs that included sawcutting a 7.5-inch groove to install a joint filler material would have resulted in cutting through dowels at the lateral contraction joints, which may have increased the potential for water to flow into the foundation. When spillway flows occur during the winter months, the crack and joint openings are likely at their widest extent. Under these conditions, the filler material is not being compressed and may be removed more easily by high velocity flows. Once removed, water can flow freely into the top half of the slab, thus decreasing the flow path to the foundation. Additionally, if filler material is removed during flow, the gap can fill up with sand and sediment that may later help to transfer temperature loads across the joint. The load transfer across the joint can lead to spalling of the concrete.

3. Using a router or hand grinder to widen major cracks over herringbone drains and then filling the gap with non-shrink grout or Sikaflex elastomeric sealant may have increased the potential for leakage into the foundation once these repairs failed. It is believed that any rigid crack filler material, such as non-shrink grout, would be subjected to high compressive stresses during summer months when the cracks close. These stresses could either crack the grout or cause spalling of the concrete adjacent to the crack. Elastomeric sealant relies on a weak bond strength to stay in place during seasonal movement of the crack and during spillway flows. Once this material fails, there is a wider crack opening for water to flow into. Filler material on a high velocity flow surface can only be considered temporary fixes, and are not a substitute for more robust repairs such as rebuilding or creating contraction joints that includes the placement of PVC waterstops.

4. Repair of surface spalls with patches will not likely resolve the spalling issue, even if the repairs extend below the rebar, and compressible material is placed over the crack or joint. As stated above, most compressible material cannot withstand high velocity flows. Once removed, the gaps can fill with other material, such as sand and sediment, which can transfer compressive loads that cause more spalling. Spalling may be detrimental, if it increases the potential for cavitation damage or leads to additional deterioration of the reinforcement by corrosion, or deterioration of the concrete such as in a freeze-thaw environment. A further problem with spalling in a high velocity flow environment, in addition to increased cavitation potential, is that it can lead to abrupt offsets into the flow path, which can result in high stagnation pressures and increased potential for flow and pressure into the foundation, unless effective waterstops are present. In the case of the Oroville Dam service spillway slab, repairs may not have been effective unless they extended the full depth of the concrete, incorporated effective waterstops, and included consideration of a temperature study to ensure that temperature stresses would not result in recurring damage.

5. Replacement of reinforcement across the cracks was not an effective deterrent to future damage. The original reinforcement likely corroded when exposed to the elements after the cracks opened. Exposed rebar at cracks can corrode, and the corrosion can work its way into the concrete where it can cause delamination. The bulky couplers used to splice the rebar can also create a greater plane of weakness in the concrete, promoting spalling. It is likely that the original single layer of #5 rebar spaced at 12 inches was not adequate for the temperature loading at Oroville. Most spillway chute slabs have two layers of reinforcement, rather than the single layer included in the Oroville spillway chute slab. However, the configuration of the herringbone drains leaving 7 inches or less of concrete cover above the drains would make it impractical to install a second layer of reinforcement in those locations.

6. It is apparent by the way that DWR has treated past repairs, that their staff did not recognize the open joints, cracking, and spalling as a source of a potential spillway chute failure mode. The repair effort included patching and sealing, both of which proved to be only temporary solutions. More substantial repairs such as rebuilding the joints and replacing concrete around drains with full-depth concrete placements and waterstops, or complete chute slab replacement never seemed to be considered before the February 7, 2017 incident, even though the repairs seemed to have a history of failing over time.

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[G-6] Email From: Author Carlton, To: Alex Samaan, Sent: September 28, 2012, Subject: Oroville Spillway Inspection 09-17-2012.

[G-7] Office Memo To: Mr. David Sarkisian, Dam Surveillance North Section Chief, From: Paul Dunlap Water Resources Engineer, Subject: Trip Report – OFD, Oro Spillway Inspection, Date: 9/26/12.

# Appendix H

# Timeline for Oroville Dam Spillways

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## TIMELINE FOR OROVILLE SPILLWAYS

*Note: The timeline provided in this appendix includes a selected list of events related to the Oroville Dam spillways. This timeline is not comprehensive, and does not specifically list all inspections of the spillways.*

- 1947 – Subsurface exploration initiated by USBR.
- 1957 to 1964 – Subsurface explorations by DWR.
  - 1962 – First interim geological report, including initial service spillway alignment.
  - May to June 1964 – Exploration test holes contract (Spec. 64-14).
  - 1964 – Second interim geological report, including final service spillway alignment.
- 1963 to 1965 – Design of spillways.
- May 1965 to February 1968 – Construction of spillways (Spec. 65-09); see Appendix A for a construction timeline.
- April 1968 – First DWR Performance Report (a total of 12 such reports were produced up until July 2006).
- 1969 – First discharges through service spillway in January and February 1969 (two separate reports in March and June 1969).
- 1969 – Regular annual inspection reports begin (FERC and DSOD).
- 1969 to 1974 – Discharges during five of the six years (none in 1972), ranging from about 61,000 to 110,000 cfs (maximum daily) for four of the years, and about 18,000 cfs (maximum daily) in 1971.
- 1973 – First FERC Part 12D inspection report, based on an inspection completed in 1972.
- 1975 to 1980 – No discharges through service spillway during these six years.
- 1977 – Repairs to spalls, “laminar cracks,” vertical wall joints,” and “expansion joints” in service spillway (Spec. 77-41).
- 1979 – Second FERC Part 12D inspection report.
- 1980 to 1986 – Discharges through service spillway during six of the seven years (none in 1985), ranging from about 38,000 to 133,000 cfs (maximum daily) for five of the years, and 13,000 cfs (maximum daily) in 1984. An intraday discharge of 133,000 cfs in 1986 was the historical peak discharge to that time.
- 1984 – Third FERC Part 12D inspection report.
- 1984 – First Director’s Safety Review Board (DSRB), Independent Consultants Five-Year Report.
- 1985 – Service spillway inspection during repairs by OFD (Oroville Field Division). Continuous crack observed across width of invert around Sta. 31+40.

- 1987 to 1992 – No discharges at service spillway during these six years.
- 1989 – Fourth FERC Part 12D inspection report.
- 1989 – Second Director's Safety Review Board, Independent Consultants Five-Year Report.
- 1993 to 1999 – Discharges at service spillway during six of the seven years (none in 1994), ranging from about 11,000 to 130,000 cfs (maximum), 130,000 cfs was in 1997; the intraday maximum discharge of 160,000 cfs in 1997 established a new record discharge, which still stands.
- 1994 – Fifth FERC Part 12D inspection report.
- 1994 – Third Director's Safety Review Board, Independent Consultants Five-Year Report.
- 1997 – Distress observed throughout service spillway, including more than 30 locations with exposed rebar. Repairs performed (Spec. 97-22).
- 1999 – Sixth FERC Part 12D inspection report.
- 1999 – Fourth Director's Safety Review Board, Independent Consultants Five-Year Report; this was the final separate Director's Safety Review Board, Independent Consultants Report, because these reports were combined with the FERC Part 12D inspection reports beginning with the 2004-2005 effort.
- 2000 to 2003 – No discharges during these 4 years, except for about 3 cfs peak (maximum daily) in 2002.
- 2004 to 2006 – Maximum daily discharges through the service spillway of 6,000 cfs in 2004, 60,000 cfs in 2005, and 74,000 cfs in 2006.
- 2004 – Potential failure modes analysis (PFMA) performed per FERC Chapter 14.
- 2005 – Seventh FERC Part 12D inspection report.
- 2005 – Memorandum from DOE Project Geology regarding right abutment rock erodibility and emergency spillway, in response to a Motion to Intervene in the FERC relicensing process for the Oroville Project.
- 2007 to 2010 – No discharges through service spillway during these four years.
- 2008 – Chain-drag survey done for service spillway chute.
- 2009 – Service spillway chute repairs, including sawcutting, spall repairs, and crack sealing (Spec. 09-14).
- 2009 – DOE erosion study performed for emergency spillway.
- 2009 – PFMA update performed.
- 2010 – Eighth FERC Part 12D inspection report.

- 2011 to 2013 – Maximum daily discharges at service spillway of about 32,000 cfs in 2011, 9,000 cfs in 2012, and 6,000 cfs in 2013.
- 2013 – Service spillway chute repairs completed by Oroville Field Division (OFD).
- November 2014 – New PFMA completed, not just an update of the prior PFMAs.
- 2014 – Ninth FERC Part 12D inspection report.
- 2014 to 2015 – No discharges at service spillway during these two years.
- 2016 – Maximum daily discharge through service spillway of about 5,000 cfs.
- January 30, 2017 – Discharges through the service spillway increased above 10,000 cfs for the first time since 2011.
- January 30 to February 6, 2017 – Discharges through the service spillway were increased in steps to about 42,500 cfs.
- February 6 and 7, 2017 – One day with discharge of about 48,000 to 50,000 cfs, while inflows increased from about 40,000 to 135,000 cfs. Reservoir elevation slowly increased from El. 850 to about 855 at time of failure.
- February 7, 2017 – Service spillway chute fails while flow was being increased from about 42,500 cfs to 52,500 cfs at about 10:00 am; after chute failure, gates were closed by about 12:30 pm.
- February 8, 2017 – Climb team inspection of damage after initial service spillway chute failure.
- February 8 through 10, 2017 – Gates opened and closed to observe severity of further erosion at the service spillway chute; discharges through the service spillway varied from 20,000 to 65,000 cfs.
- February 10, 2017 – Discharge through service spillway maintained at 65,000 cfs for about 17 hours, then reduced to 55,000 cfs at about 8:00 pm.
- February 10 through 12, 2017 – Discharge through service spillway maintained at 55,000 cfs from about 8:00 pm, February 10 until about 3:35 pm on February 12.
- February 11, 2017 – Water began to flow over the emergency spillway for the first time, sometime between about 7:00 and 8:00 am.
- February 12, 2017 – Emergency spillway discharge peaks at about 12,500 cfs, 1.6 feet of flow over the spillway, at about 3:30 pm. Rapidly advancing headcut erosion observed downstream of the emergency spillway crest structure; evacuation order issued at about 3:44 pm; service spillway gate openings increased starting at about 3:35 pm; discharge through service spillway reaches about 100,000 cfs by about 7:00 pm.
- February 12 through 16, 2017 – Spillway discharge maintained at about 100,000 cfs from about 7:00 February 7 through 8:00 am February 16.

- February 14, 2017 – Evacuation order changed to an evacuation warning at about 3:30 pm.
- February 16 through May 19, 2017 – Service spillway gates operated to maintain reservoir at or below Elevation 850, which was first reached on February 20, with some periods when the gates were closed to allow for gathering of information for development of repair plans.
- March 19, 2017 – Evacuation warning rescinded.
- May 19, 2017 – Service spillway gates closed for the season to allow for initiation of spillway repair construction.

# Appendix I

# Oroville Dam Service Spillway Damage Forensics

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## TABLE OF CONTENTS

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## 1.0 BACKGROUND

On February 7, 2017, a portion of the service spillway chute failed downstream from Station 33+00 during spillway operation. The extent of the damage as observed after the spillway flows were shut down extended laterally across Lanes 2 to 5 of the spillway chute. The spillway lanes are numbered right to left looking downstream, with Lanes 1 and 6 being comprised of the chute wall bases, and Lanes 2 through 5 being the chute slabs. Each of the chute slab lanes are 40 feet wide. The damage extended the furthest downstream in Lane 5. Figure I-1 shows the extent of the damage to the spillway chute after the spillway gates were closed that day. Due to high inflows from the ongoing flood, the spillway gates were reopened the following day. Continued spillway flows in the following weeks resulted in additional damage to the service spillway as flows into the exposed spillway foundation continued to erode more of the spillway chute. This appendix addresses the conditions and causes of the initial service spillway chute failure on February 7, 2017.


**Figure I-1: Damage, February 8, 2017**

## 2.0 EYEWITNESS ACCOUNTS

The IFT (Independent Forensic Team) interviewed several eyewitnesses to the spillway damage. These eyewitnesses were on site on February 7, 2017 at various times and at different locations as the damage progressed. None of these eyewitnesses observed the initial failure, so it was not possible to pinpoint the location of failure initiation. The IFT reviewed the many photos and videos taken by these eyewitnesses to help understand how and why the chute failed. Figure I-2 [I-1] was modified to show spillway stationing and approximate locations of these individuals (text boxes with time of photo). Figures I-3 through I-9 show the damage at successive times.

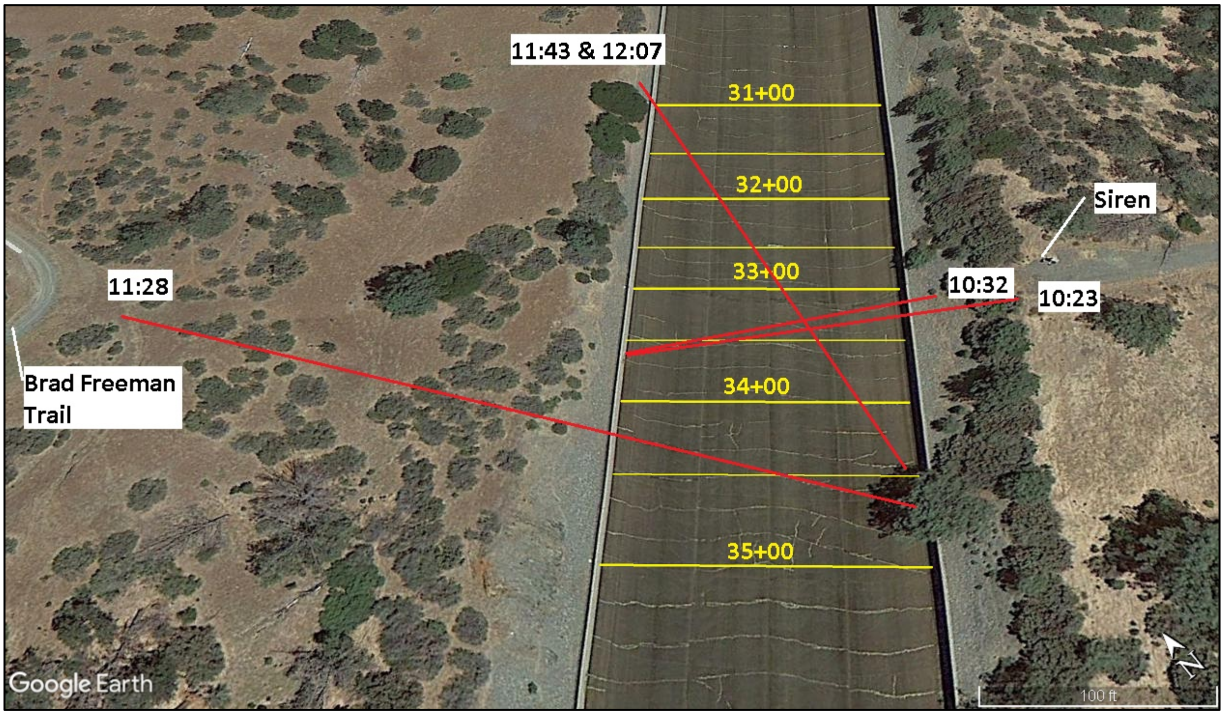


**Figure I-2: Location of Photos in Figures I-3 to I-9 Based on Time of Photo [I-1]**

On February 7th, at approximately 10:10 am, DWR maintenance staff workers, who were servicing the siren on the left side of the spillway, next to the spillway chute, heard what they said sounded like an explosion or a loud bang, and saw spray coming from the spillway chute. One of the workers began to take photos and videos [I-2]. The first video was made at 10:23 from the location noted on Figure I-2, and two still frames from that video are shown on Figures I-3 and I-4. They reported that the damage appeared to start near the center of the chute and then move toward them (the left wall.) As they watched, the area of spray also moved upstream. A photo taken at 10:32 am shows that the damage is occurring downstream of Station 33+00, Figure I-5.

At 11:28 am, two hikers on the Brad Freeman Trail observed water shooting above the chute. They walked to a gully about 275 feet from the chute on the left side of the spillway and photographed the flow, Figure I-6. They also took the video that was published on YouTube.com *Lake Oroville dam spillway disaster. First onsite!* [I-3]. They had tried to get closer to the chute, but were stopped by DWR personnel for safety reasons.

Shortly after the hikers took the photo, two trucks from DWR arrived at the right side of the spillway where the damage could be observed. One of the passengers started videotaping the event at 11:42, Figure I-7 [I-2].

Later, one of the original DWR maintenance staff workers assigned to service the siren also took additional videos of the spray in the chute, from the right side of the spillway just before the spillway gates were closed, Figure I-8 [I-2]. This shows that the damage has progressed upstream to Station 33+00.

The shut down of spillway flow began at a about 11:25 am and the gates were fully closed by about 1:00 pm. Figure I-9 [I-2] shows flow on the spillway chute at 12:26 pm, when it appears that the gates are nearly closed. Note that spray from a failed repair or spall can be seen upstream of Station 33+00. This indicates that previous repairs had probably failed within the chute. This occurrence is discussed further below.

Figures I-3 through I-9 show the flow in the chute at successive times.

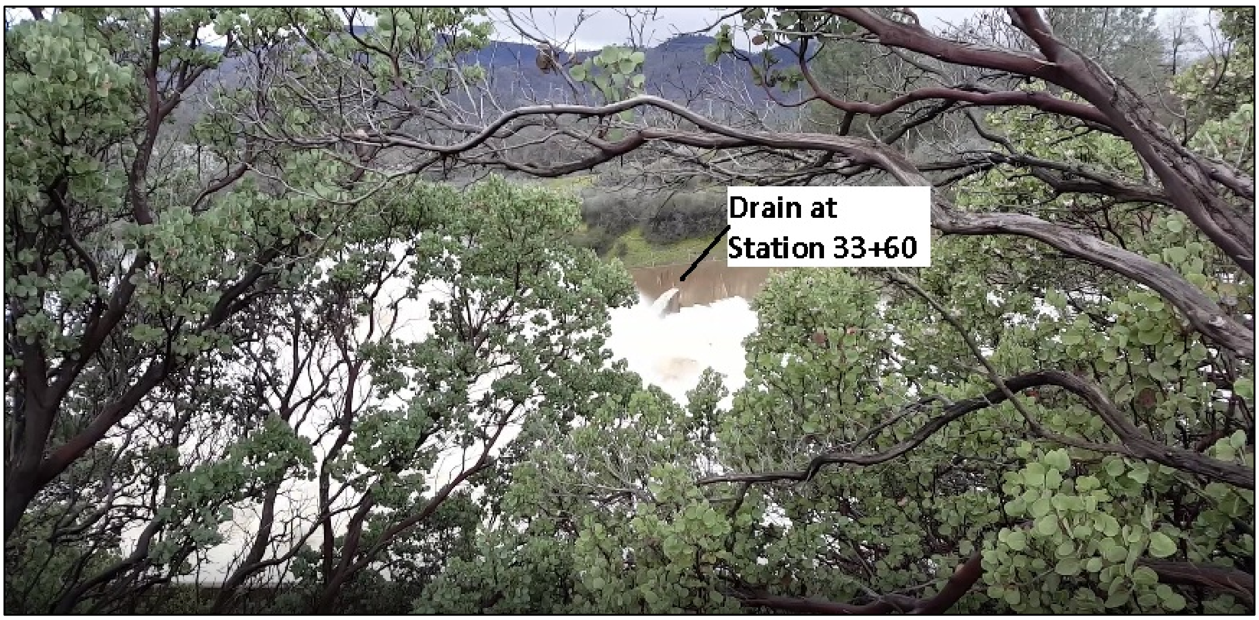


**Figure I-3: Flow in Chute at 10:23 am [I-2]**


**Figure I-4: Flow in Chute at 10:23 am, a Few Seconds Later [I-2]**


**Figure I-5: Flow in Chute at 10:32 [I-2]**

**Figure I-6: Flow in Chute at 11:28 [I-3]**


**Figure I-7: Flow at 11:43 [I-2]**


**Figure I-8: Flow in Chute at 12:07 [I-2]**


**Figure I-9: Flow in Chute at 12:26 [I-2]**

## 2.1 Conclusions from the Photos

A careful examination of the photos shows that the damage likely started downstream of Station 33+50 in Lanes 3, 4, or 5, as seen in Figure I-5. This damage is believed to be the result of a loss of a portion of the invert slab, and not an entire 40- by 50-foot chute slab panel. The initial failure could have been as small as a one square foot patch or spall above a drain, or as large as a 20-foot section between the cracks above the herringbone drains. Once this initial portion of the slab failed, it likely triggered a chain of events, resulting in additional slab section failures. The plume of water from the larger damage area spilled over the left chute wall and eroded some of the material behind the wall.

Damage may have initially occurred downstream of Station 34+50, but the photographic evidence is not sufficient to support this interpretation. If damage occurred this far downstream, the damage quickly moved upstream of Station 34+00. This is plausible if the removal of chute slab downstream of Station 34+50 resulted in upstream slab portions sliding downstream due to lack of downstream buttressing and poor foundation conditions. The flow in Figures I-7 and I-8 show that the damage has moved upstream to Station 33+00 by the time these photos were taken.

Sometime during this process, the portion of the slab downstream of Station 33+00 was lost by first sliding downstream and then being removed by hydraulic jacking as a gap opened at the contraction joint location. Observations from photos taken by the climb team that entered the erosion hole on February 8, 2017 and drone video taken that day show that the shear key in the contraction joint at Station 33+00 had not failed, which would have been necessary if the slab had lifted up at that location. The intact, overlapping key upstream of Station 33+00 can be seen in Figure I-17. This, photographic evidence indicates that the portion of the slab just downstream of Station 33+00 could not have been removed first by hydraulic jacking, and had to have either moved downstream or have dropped into an eroded area beneath the slab, before being removed by the flow.

A portion of the chute slab upstream from Station 34+00 failed by rotating about the downstream edge as can be seen in Figure I-10, where the rebar is bent downstream. It can also be seen from this photo that slab sections downstream from Station 34+00 on either side also rotated out, bending the rebar downstream. It appears that the failed upstream slab portions failed by lifting of the upstream edge of the slab and rotating about the downstream edge.


**Figure I-10: Initial Damage Area Looking Downstream Showing Bent Bars Upstream of Sta. 34+00**

## 3.0 SPILLWAY DESIGN, CONSTRUCTION, AND REPAIRS

Spillway chute design and construction are discussed in Appendix A of this document. While the IFT believes that there is no single factor that led to the chute failure on February 7, 2017, there are some factors related to the design and construction that played a key role in the failure of the spillway chute. In Appendix E of this document there is a comparison of spillway chute design details from more than 100 other projects completed in the general time period when the Oroville Dam spillway was designed and constructed. Design, construction, and repair issues played a critical role in the chute failure. These issues include the contraction joint spacing and configuration, herringbone drains that were embedded into the concrete slab, a single layer of reinforcement, a relaxation of foundation requirements, and subsequent repairs that were not reliable.

### 3.1 <u>Joint Details</u>

In general, it was not common practice in the early 1960s to provide waterstops at chute joints. While the IFT believes that waterstops at chute joints, which are now common practice, would have helped at Oroville, there are many older spillways that were designed without waterstops prior to the 1970s, which have operated without incident related to the waterstop issue. Instead of providing waterstops at contraction joints, it was common practice for some design organizations to provide offsets at the contraction joints such that the downstream side of the joint was lower than the upstream side to prevent an offset into the flow. This design feature also prevents the formation of stagnation pressures at the joints due to a projection of the downstream edge into the flow. Offsets joints were specified at the contraction joints at Oroville as well, but is believed that that this detail may have only been provided at 400-foot spacing. However, based on drawing details, the IFT believes that the contraction joint at Station 33+00 had this detail. Oroville contraction joints with this design feature could generally be expected to perform well if the as-built conditions could be maintained. However, without waterstops, these joints can be vulnerable to stagnation pressure formation, if there is spalling at the upstream side of the joint, creating a vertical face at the downstream joint surface that projects into the flow.

The chute has two basic types of lateral joints. Formed lateral contraction joints were to be constructed every 100 feet along the chute length, and intermediate joints were to be created by forming a groove in the concrete surface after slip forming (see Appendix A). The formed contraction joints were keyed by providing an offset 7.5 inches below the surface. The offset resulted in the top 7.5 inches of the upstream slab overlapping the downstream slab such that the upstream end of the downstream slab cannot move upward without either lifting the upstream slab or shearing through the key. Intermediate joints observed in the field at 50-foot stationing along the spillway chute did not appear to be formed. Instead, the surface was treated by cutting a shallow one inch groove during construction to promote crack initiation (much like a sidewalk groove). It is believed that the reinforcement is continuous across these grooved joint locations, based on construction photos, making these joints control joints. However, physical evidence suggests that many of the intermediate joints never fully formed, because cracking occurred at herringbone drain locations, where more than half the concrete section was eliminated by drain placement, rather than at the thicker section below the shallow groove. Opening of the cracks over drains has been significant, and because of their spacing (20 feet in the steep chute section), they likely take up much of the temperature related movement.

## 3.2 Drainage

At some point after the designs were complete and the bid specifications were issued, the drawings were revised to include larger drainage pipes, both for the herringbone drains and the collector drain pipes (see Appendix A). The geologists had anticipated groundwater seepage from the foundation, and the drainage system design was based on the observed seepage at the site. The amount of flow that enters the drainage system from within the spillway chute was not anticipated.

The herringbone drain pipes were to be embedded into the bottom of the slab, rather than being placed entirely below the slab, as was done with underdrains by many other design organizations at the time Oroville was being constructed (see Appendix E). This was likely due to a concern that blasting trenches in the rock for drains would cause excessive damage. The change from 4-inch herringbone drains to larger 6-inch herringbone drains reduced the theoretical concrete cover over the drainage pipes to 7 inches. With only 7 inches of concrete cover, less than half of the design slab thickness of 15 inches is provided over the drains, making this reduced concrete area a prime location for cracking. Cracks formed over the herringbone drains shortly after construction. These cracks open significantly as the concrete chute slab contracts in reaction to low temperatures. This cracking and subsequent opening made the intermediate grooved joints at 50-foot stationing ineffective, since stress relief was occurring at the drain locations.

Additional details of the drainage, including estimated drainage capacity, is discussed below as contributory factors to the failure.

### 3.3 Foundation Conditions

As discussed in Appendix A, foundation preparation and cleanup was specified to be the same in the spillway chute as it was in the headworks structure. The specifications paragraphs made no distinction between the chute and headworks in terms of the level of effort to prepare and clean the foundation prior to placement. However, during construction, as the contractor moved to the steep section of the chute, the quality of the foundation at the specified grade, did not seem to meet the designer’s expectation. It would require significantly more excavation and backfill concrete, and delays to the contractor construction schedule to achieve the specified foundation. Although the IFT could not find adequate documentation related to the decision to relax the foundation requirements during construction, construction photos and inspection reports summarized in Appendix A show that the design intent of a hard, slightly weathered foundation was not being met. Some of the worst foundation conditions in the entire chute were at the location where failure initiated (see Figure I-11).

From construction photos and reports (Appendix A), it appears that much of the foundation in the vicinity of the initial failure was covered by soil-like materials. The weathering profile and presence of shear zones likely contributed to this. To obtain an adequate foundation downstream of Station 33+00, the IFT believes that significant local overexcavation may have been necessary, but construction records do not indicate that this was requested of the contractor during construction. The post-damage photos in this appendix and elsewhere in this document show that rock below the foundation contact significantly improves at the downstream end of the failed area, where shallow scour depths can be seen. However, cleanup of the foundation had been an ongoing concern by the time the contractor reached this area (work moved from upstream to downstream), and in an effort to limit claims DWR staff seemed to not be directing the contractor to overexcavate the foundation, but rather clean it up the best they could when they reached the design grade without going deeper. By comparing Figure I-11 to Figure I-20, it appears that a good clean rock foundation could have easily been achieved downstream of Station 33+50 with little more effort (based on shallow scour depths to good rock), but apparently the effort to limit overexcavation and backfill concrete also resulted in a relaxation in the specified cleanup effort in areas like this where it could have provided significant benefit.

From the original geologic data, it could have been anticipated that local overexcavation would be necessary to achieve the specified foundation conditions. The depth of erodible foundation material immediately downstream from Station 33+00 is significant, and as discussed in Appendices A and C, the specified quantities for overexcavation and backfill concrete may not have been sufficient to cover all of the areas requiring overexcavation. However, this is one area that the IFT believes should have been overexcavated.

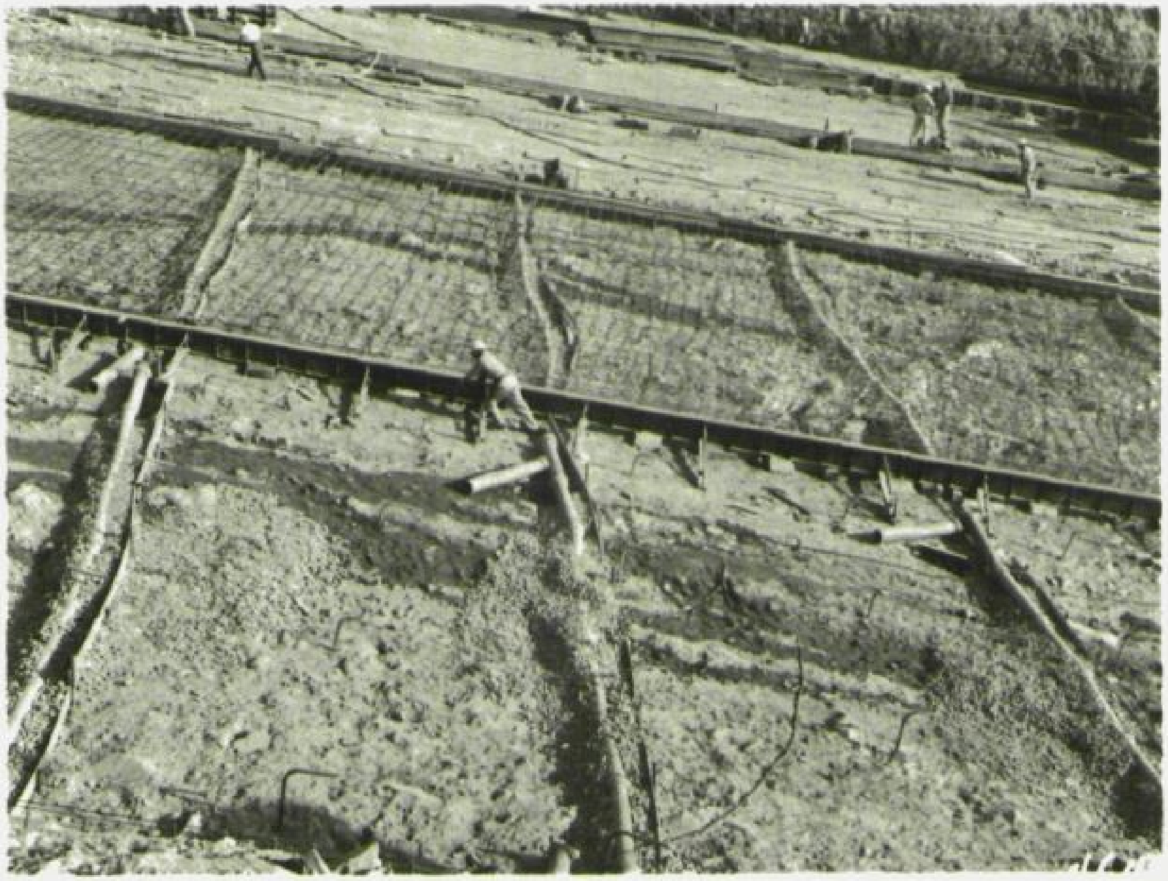

Photo 39. Chute foundation in vicinity of Sta. 33+60. Tile and gravel underdrains in lanes 2 and 3, rebar in lane 3. View southeast.
Neg. No. 4644 11-2-66

**Figure I-11: Foundation in the Vicinity of the Failure [I-4]**

## 3.4 Foundation Anchors

Anchor bars were to be grouted 5 feet into the foundation. The discussion of anchor bar strength indicates that the anchors were designed for 5 feet of uplift. The anchor bar embedment was to be tested to develop 30,000 psi stress in the No. 11 bars. These tests were conducted in the upper portion of the chute when construction of the chute was just underway. There was no retesting of this embedment length in the steep portion of the chute, where foundation conditions were not as favorable as in the upper chute. Based on descriptions of the foundation and photographic evidence in Appendix A, it is believed that many of the anchors in the area of the initial chute failure were embedded in a much weaker foundation than in the areas where anchors were tested, especially where the deep erosion occurred on February 7 immediately downstream from Station 33+00. Some of these anchors may have developed very little pull-out strength.

## 4.0 PRIOR REPAIRS

Documented spillway repairs were made at various times between 1977 and 2013. Appendix G provides a detailed discussion of these repairs. In 1977, it was reported that repairs included 10 damaged areas along the Sta. 33+00 contraction joint. The repairs included patching of spalled or delaminated concrete and a 7.5-inch deep by ⅜-inch wide sawcut at the top the contraction joint to replace expansion material. This sawcut severed the dowels installed at the joint.

In 1985, repairs were made at Sta. 33+00 and 34+50; however, no documentation was found concerning the number and location of these repairs. Repairs made in 1997 included 18 areas between Stations 33+00 and 35+00. Repairs to lateral and longitudinal joints and lateral cracks were needed in this area.

In 2009, extensive repairs were made to the spillway [I-5], including repairs made upstream and downstream of Station 33+00, as shown in Figure I-12. Many of the cracks over herringbone drains and associated spalls were repaired in 2009. In many locations, cracks over the drains had opened enough to fail the reinforcing bars crossing the crack. It is believed that high tensile stress and weakening of the rebar by corrosion caused the failures. Mechanical couplers were used to splice the broken reinforcing bars back together, and a crack inducer was sawcut into the slab above the drains.

While there were several new and existing patches to spalled or delaminated areas near the Station 33+00 joint and between Station 34+00 and 35+00 in 2009, there were fewer new and existing patches between Station 33+00 and 34+00. It is not known if weaker foundation and poor anchor strength between Station 33+00 and 34+00 resulted in higher stress concentrations upstream and downstream from this section where the repairs were more concentrated, or if there was a similar amount of delamination damage but just fewer repairs of the damage in this area.

Along with patches near cracks and joints, the 2009 repairs included attempts to seal the cracks (heavy lines in Figure I-12). Cracks on the right side of the chute (left half of the figure) were reportedly grouted with non-shrink grout, while the left side cracks were reportedly filled with Sikaflex elastomeric sealant.

Further repairs were made in 2013, as shown in Figure I-13. These repairs seem to included repairs to areas with previous repairs that had failed and new areas of damage. Repair methods were similar to the 2009 repairs [I-5], although it is believed that non-shrink grout was not used in 2013. The 2013 repairs in the failed area between Station 33+00 and 34+00 were again, mostly sealing of cracks, with few new patches. While it is assumed that there were few visible spalls between Station 33+00 and 34+00 in 2013, it is not known if there were delaminations developing in that area that were undetected at the time. A possible explanation for fewer patches in this area could be that the chute slab was partially founded on a better foundation downstream from this area, as is evident from the post-failure photos showing shallow scour depths and intact slab sections further downstream. As discussed above, the weaker foundation between Station 33+00 and 34+00 may have allowed the slab to move more freely, causing stress concentrations upstream and downstream.

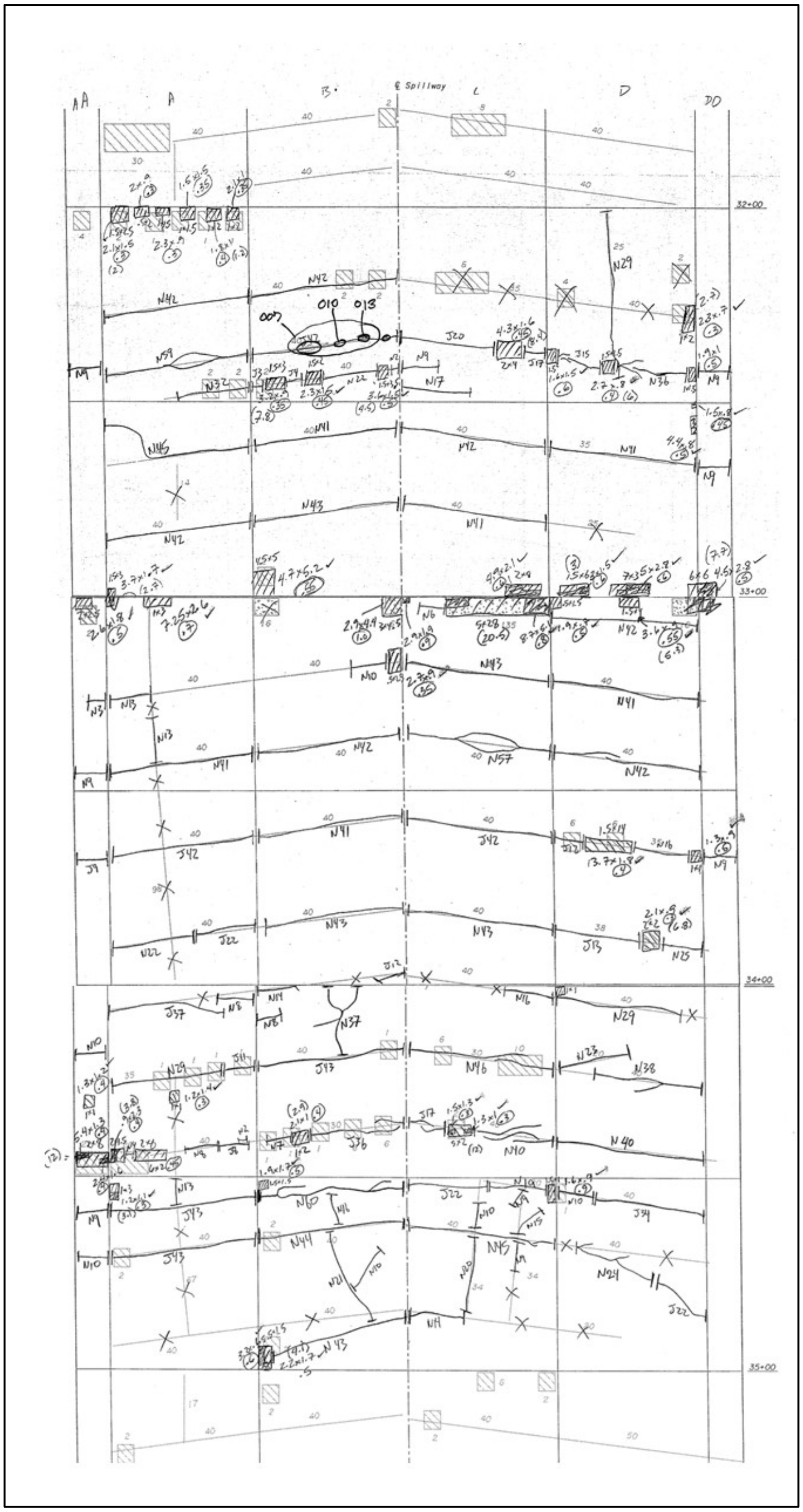

**Figure I-12: 2009 Repairs [I-5], Sta. 32+00 to 34+00 from Field Notes**

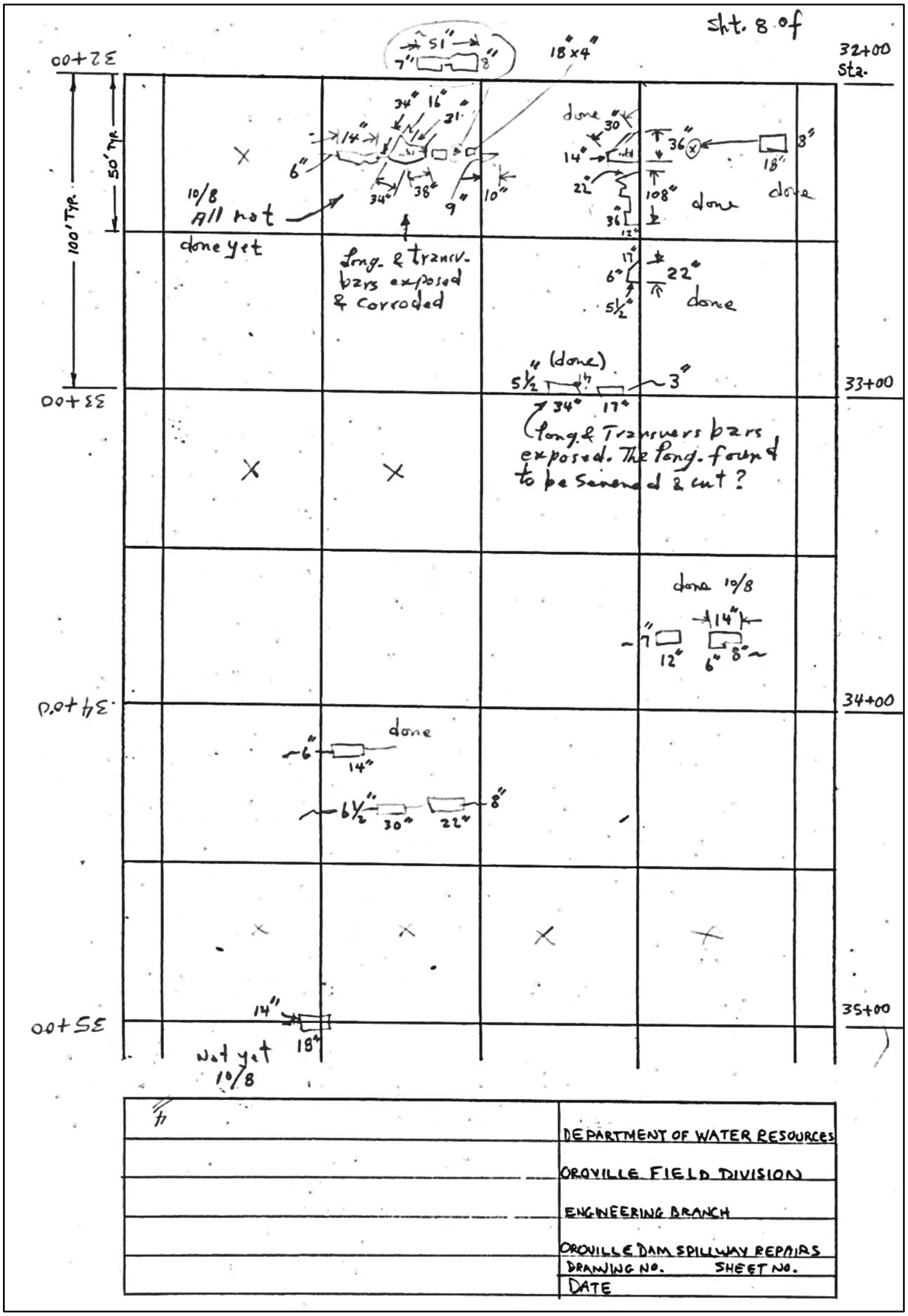


**Figure I-13: 2013 Spillway Repairs from DWR Staff Field Notes from Sta. 32+00 to 35+00**

As discussed in Appendix G of this report, repairs to the spillway chute slab made prior to the February 7, 2017 incident were ineffective long-term solutions to an ongoing problem. It is unlikely that cracks over the herringbone drains could be prevented from opening and closing during seasonal temperature cycles, and the repairs were not robust enough to prevent occurrence of further damage within a few years of repair completion. The sealing of cracks and contraction joints was done with materials that could not withstand these seasonal cycles, and/or would not remain in place during high velocity spillway flows. While the concrete repairs patched over spalled concrete, the patches were relatively shallow, and were not capable of preventing new spalls in either the repaired concrete or concrete that had not previously spalled. Spalling at cracks and joints creates a significant potential for water to flow into the foundation and generate high uplift pressures, if the drains cannot successfully remove these inflows. Two critical design details that were never addressed by any of the repairs are the lack of waterstops at the contraction joints, and the herringbone drains being placed within the chute concrete where they create a weakened section that can easily crack and indeed had done so throughout the chute.

## 5.0 INCIDENT TIMELINE

The spillway has operated in the past with discharges larger than the one for which failure occurred in the past, as shown in Figure I-14.

Although repairs were made as recently as 2013, it is believed that significant flood releases that began in late January 2017 had damaged some of the previous repairs, and possibly caused damage to new areas of the chute that were not previously damaged. While normal seasonal temperature cycles could have compromised and/or damage the repairs, additional damage likely occurred as a result of the high velocity flows in the spillway chute during the spillway operations leading up to the chute failure incident of February 7, 2017. Photographic evidence of high collector drain flows prior to February 7 is an indication that water flowing in the spillway chute was reaching the foundation, where it was discharged through the herringbone drainage system into the collector drains. This can be seen as drain flows changed from January 13, to January 27, 2017 (Figures I-15 and I-16, respectively). Figure I-16 also shows a drain set that did not flow on the left side of the chute and another drain set that flowed intermittently on the right side of the chute.

While some minor flow in the drains can be expected to be from groundwater seeps and surface runoff into the gravel surrounding the collector drains (possibly seen as minor seepage on January 13), the increase in the observed flows on January 27 may have been caused by additional leakage from the chute into the foundation. Even though similar gate leakage is flowing down the chute in both photos, only very minor spillway flows occurred prior to the January 13, 2017 photo, while more significant flows of up to 9,700 cfs occurred between the dates of the two photos. Since both photos were taken on sunny days, surface runoff is not considered a significant contributor to the drain flows. One possible explanation for the increased flows is that the number surface features in the spillway chute allowing flow to enter the foundation has increased between the two dates because of spillway flows causing a number of previous repairs to fail, along with new damage to the chute. However, as discussed in Appendix F1, drain flows have been previously documented to significantly vary within a few days under apparently identical conditions, likely due to time lags following spill and rainfall events.

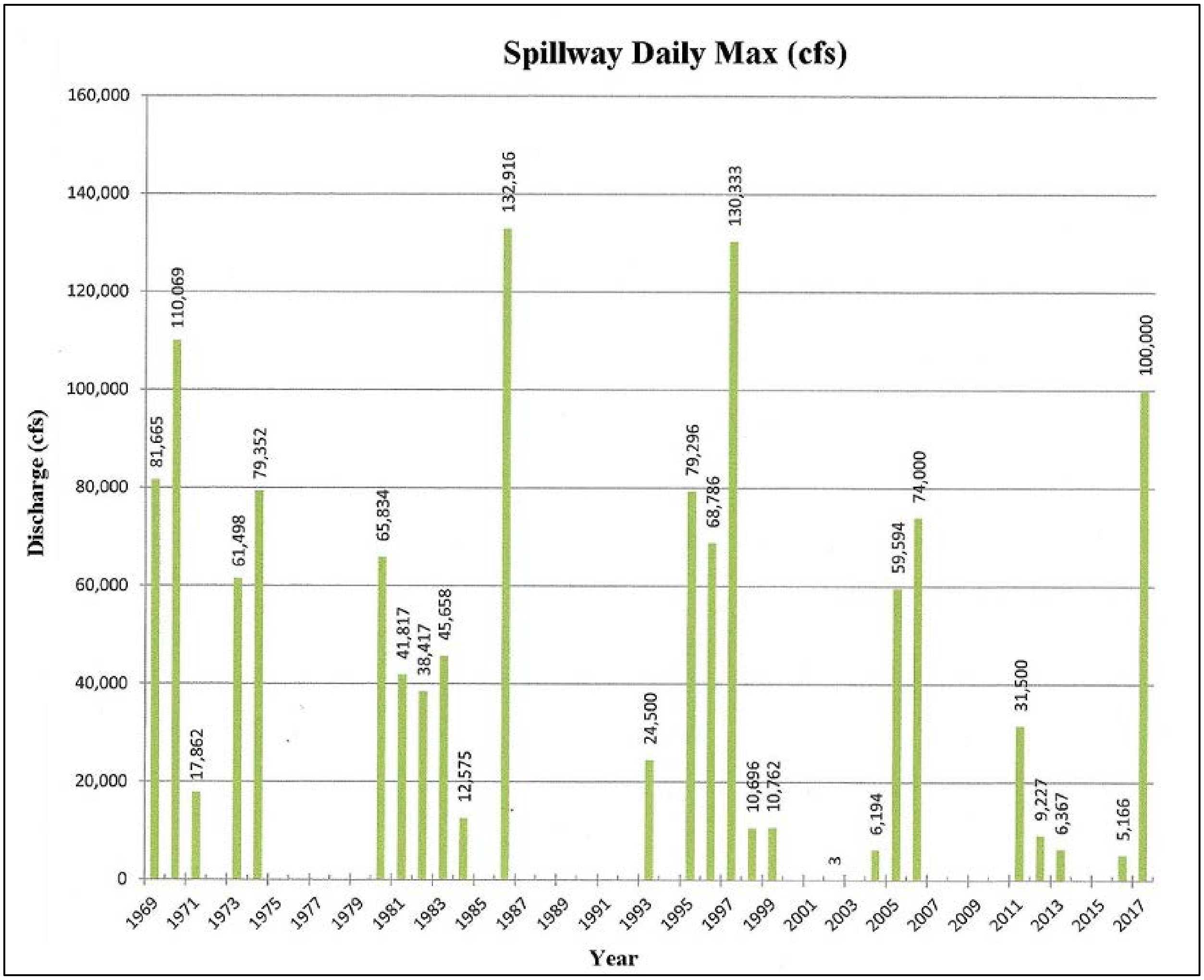


**Figure I-14: Historic Daily Maximum Flows (chart provided by DWR)**

On January 13, 2017 during low flows in the spillway caused by gate leakage, a disturbance in the water surface was observed in Lane 5 at what is believed to be Station 33+00 (see figure I-15). While the feature causing the disturbance cannot be identified from the photo, it is reasonable to assume that this is new damage or a failed repair since 2013, when the last repairs were completed on the spillway chute. Repairs to the Sta. 33+00 contraction joint in Lane 5 were last documented in 2009 (see Figure I-12). It can also be observed that this damage does not seem to be causing increased drain flow from the outfall along the spillway wall (right side of photo) at that time.

**Figure I-15: Flow Disturbance in Lane 5 on January 13, 2017 [I-6]**

On January 27th, 2017 (Figure I-16), the area of the flow disturbance seems to be caused by the same repair failure or spall in Lane 5 at Station 33+00 as shown in Figure I-15. Note that patches in Lane 4 can also be seen, but appear to be intact. If this is a spall or failed patch in Lane 5, and it is upstream from the joint, it could create a stagnation point with a vertical offset into the flow at the joint, where water could flow into the joint. However, if the spall is downstream, the offset would be several inches downstream from the joint, and the path to the foundation would not likely be through the contraction joint at Station 33+00, but could instead be through a crack over the herringbone drain that is located a short distance downstream from the joint.

**Figure I-16: Spall in Lane 5 Jan 27, 2017 [I-6]**

Figure I-17 is a still shot captured from drone video taken on February 8, 2017 after the initial damage occurred. Note that the patch upstream from the joint in Lane 5 is not damaged. This photo is evidence that the upstream patch did not fail before the chute failed. This leaves two other possible explanations for the flow disturbance as seen in Figures I-15 and I-16. The first is that a failed older patch or new spall occurred downstream from the joint. The second is that the flow disturbance is caused by joint filler material that has been partially removed from the joint. There is no clear indication from past repair reports that a repair was ever made at this location downstream from the joint, but it is possible that repairs were made that were not documented in available information. The IFT could not obtain a clearer copy of the Figure I-16 image to help identify the cause of the disturbance.

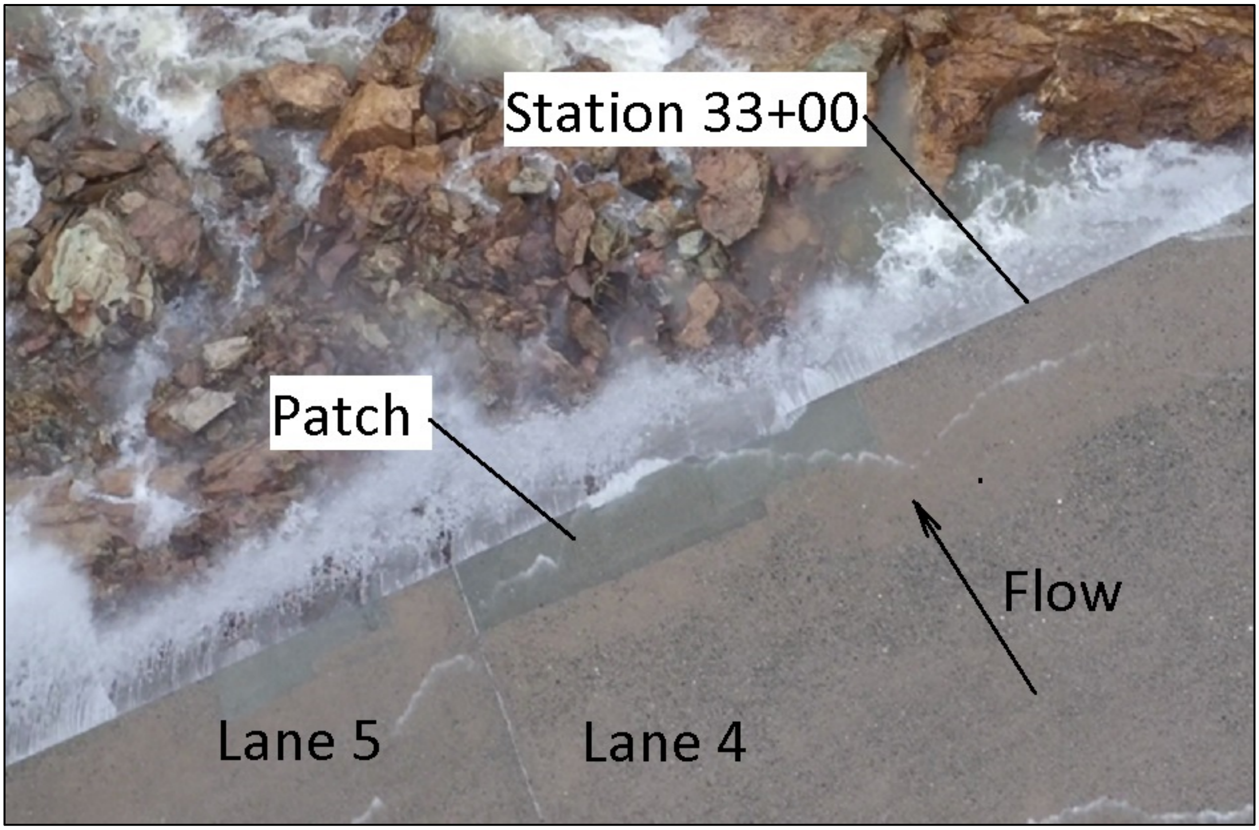


**Figure I-17: Patches Upstream from Sta. 33+00 [I-2]**

The cracks over the herringbone drains are clearly visible from a satellite photo taken on July 9, 2010, Figure I-18. As can be seen in Figure I-18, a herringbone drain crack is located a short distance downstream of the joint at Station 33+00 in Lane 5. The feature in Lane 5 (Figure I-16) could be a failed section of slab over the drain where the concrete is thinner. Cracks that form over the herringbone drains have been known to cause a rupture of the rebar. This could be a failed section of concrete bounded by the contraction joint at Sta. 33+00 upstream and the crack over the drain downstream. Since dowels were cut at this contraction joint when repairs were made in 1997 (see Appendix G), a section of concrete bounded by the joint upstream and a drain crack with failed rebar on the downstream side may only be held in place by rebar that is parallel to the lateral joint. It cannot be determined if the concrete has been removed at the time of the photograph, it may still be hanging on by a single rebar passing through the loose concrete. However, with no clear photos made available to the IFT, actual conditions at this location prior to failure remain unknown.

On February 7, 2017, extensive damage occured downstream of Station 33+00 after the flow was increased from about 42,500 cfs to 52,500 cfs at about 10:00 am that morning, as shown on Figure I-1. The point where zero flow occurs on the February 7 (Figure I-19) is the time when the spillway gates had been closed after the damage was discovered on February 7.


**Figure I-18: Herringbone Drains Visible from a Satellite Photo (highlighted in blue) [I-1]**

The damage downstream of Station 33+00 is shown in a February 8 photo (Figure I-1). This figure also shows the drain set that did not flow on the left side of the chute (the outfall for the herringbone drains from 31+00 to 33+00, opposite the drain seen flowing on the right side) and the drain set that flowed intermittently on the right side of the chute (the outfall for the herringbone drains from 29+00 to 31+00). A close-up of this damage and approximate spillway stationing can be seen in Figure I-20.

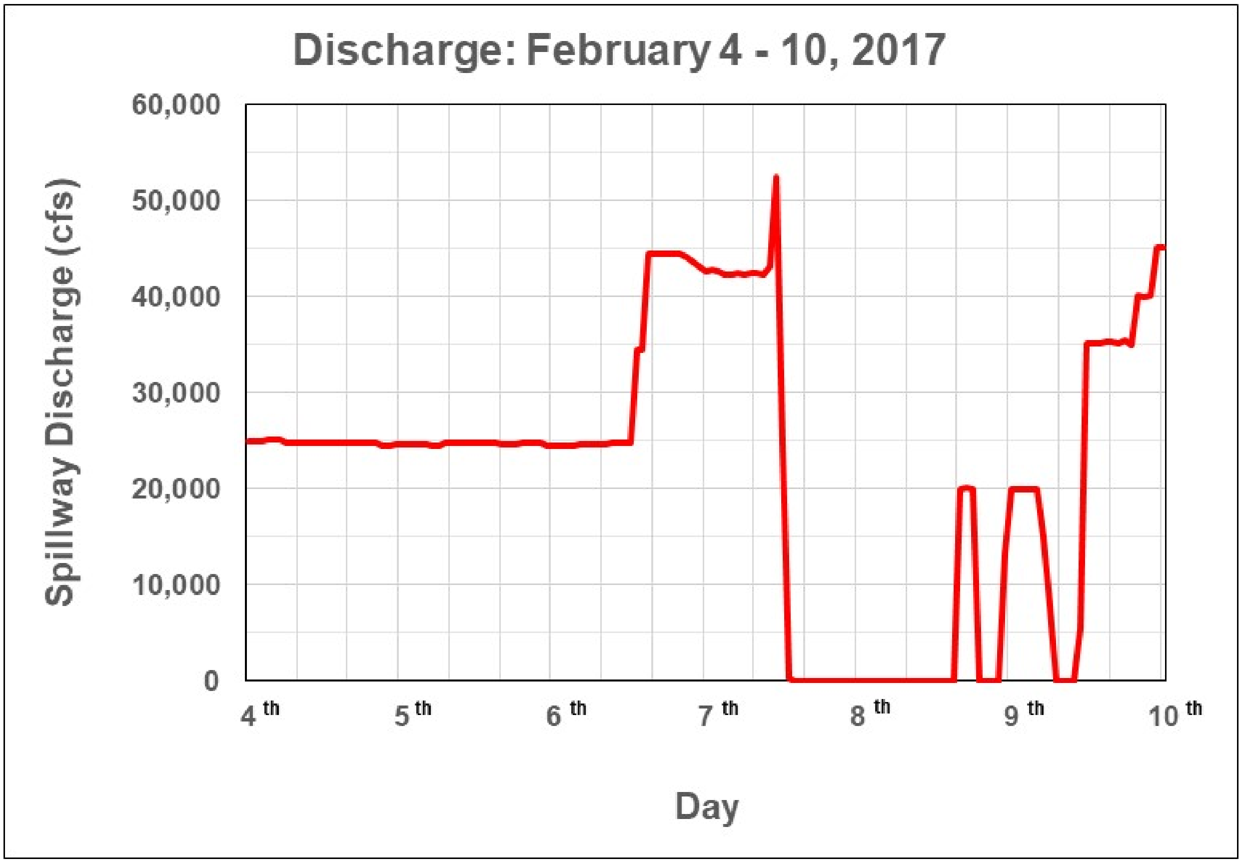


**Figure I-19: Spillway Flow History, February 4 to 10, 2017 (data provided by DWR)**

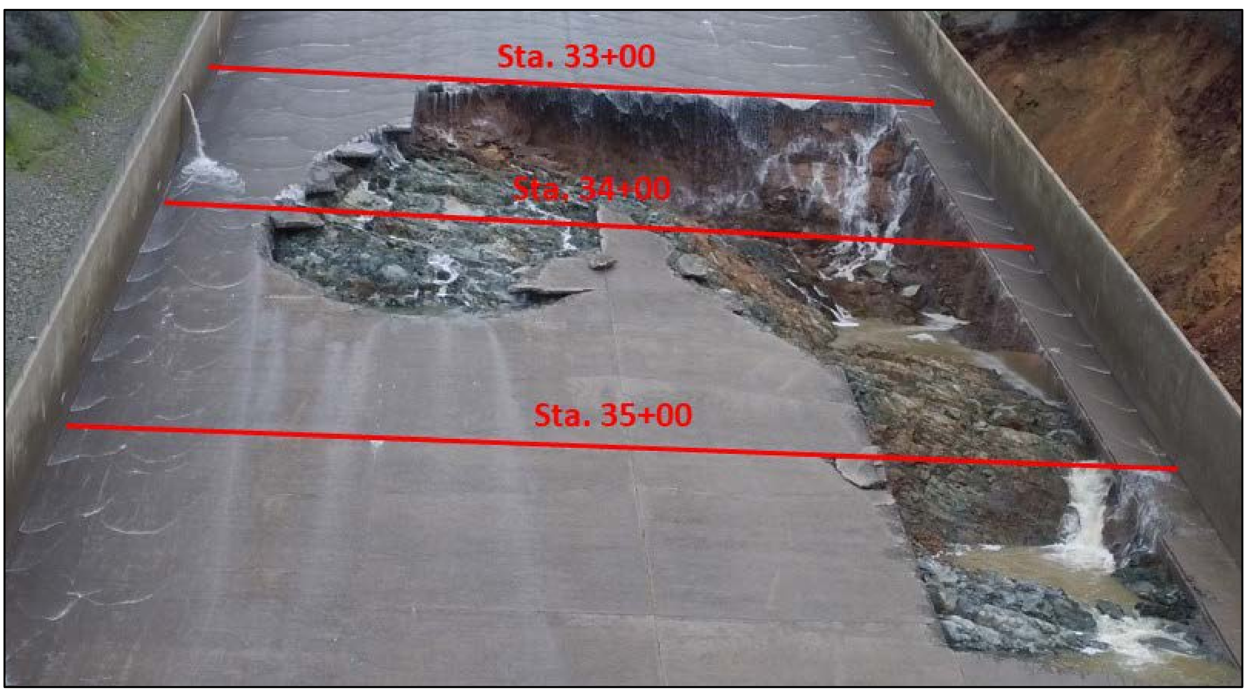


**Figure I-20: Closeup of Damage on February 8, 2017 [I-2]**

## 6.0 CONTRIBUTORY FACTORS LEADING TO FAILURE

As discussed above and in Appendix E, the spillway design details lacked some common defensive measures used in modern spillway construction which would help prevent a chute failure. The repairs made to damaged areas of the spillway chute were also not robust enough to withstand high velocity flows, as discussed in Appendix G. Other contributory factors include the drainage system limitations and the foundation providing poor anchorage in the area where the failure initiated.

### 6.1 Drain Details

As discussed above, the drainage was designed for anticipated ground water seepage beneath the spillway chute. Collector drains collected flow from "sets" of herringbone drains. The size of the collector drains was increased during construction to account for additional flow, anticipated as runoff into the drainage envelope outside the chute walls that surrounded the collector drains.

Drainage near Station 33+00 and the damaged that occurred on February 7, 2017 can be seen in Figures I-1, I-16 and I-27. While the collector drain outfall on the right side of the spillway near Station 33+60 (left side of the photos) is flowing, the corresponding drain on the left side was not flowing. That collector drain was washed away on the back side of the wall during the initial chute failue, as seen in Figures I-20 and I-27. The lack of drainage from the drain outfall near Station 33+60 on the left wall has been attributed by others to tree roots in the collector drains. Refer to Appendix F1 for further discussion on this issue. It is also possible that either this drain was not connected to the outfall pipe prior to the damage, and flow from the herringbone drains discharges into the gravel outside the wall, or there was actually no inflow to the herringbone drains in that drain set because the repairs there had not yet failed, and no flow was entering the foundation. At any rate, the drain that was not flowing on the left side services the area upstream of Station 33+00. The drain outfall for the area downstream from Station 33+00 can be seen flowing in pre-failure photos, such as the full photo from which Figure I-16 was taken.

Another possible explanation for no drain flow at Station 33+60 is that the water from the chute flow surface entered the foundation, bypassed the herringbone drains by flowing under the slab through piping (internal ersosion) channels downstream to the next drain set. This could happen if the slab is thick enough in this area (upstream from Sta. 33+00) that the drains are well above the foundation, and water seeping into the foundation passes through a thick gravel layer placed beneath the drains where the foundation was overexcavated, making its way to the downstream drainage set that is in the area of the initial damage. If this happened, then the next downstream drain set may not have sufficient capacity to handle the combined drainage flow and that downstream drain set from Station 33+00 to Station 35+00 could have reached capacity causing flows to back up under the slab.

The light orange shaded areas shown in Figure I-21 are strongly weathered foundation materials. These materials are in an area of very weak and erodible foundation material. This material was present in the foundation from Sta. 33+00 to 35+00. Over time some of this material could have experienced erosion from water flowing into the foundation from the chute surface, creating void areas under the slab where water could accumulate once the drain capacity is exceeded. The IFT believes that the voids would have been relatively shallow, because the material included fines and rock fragments. Once fines are removed, the rock fragments would tend to armor the surface, preventing deep erosion.

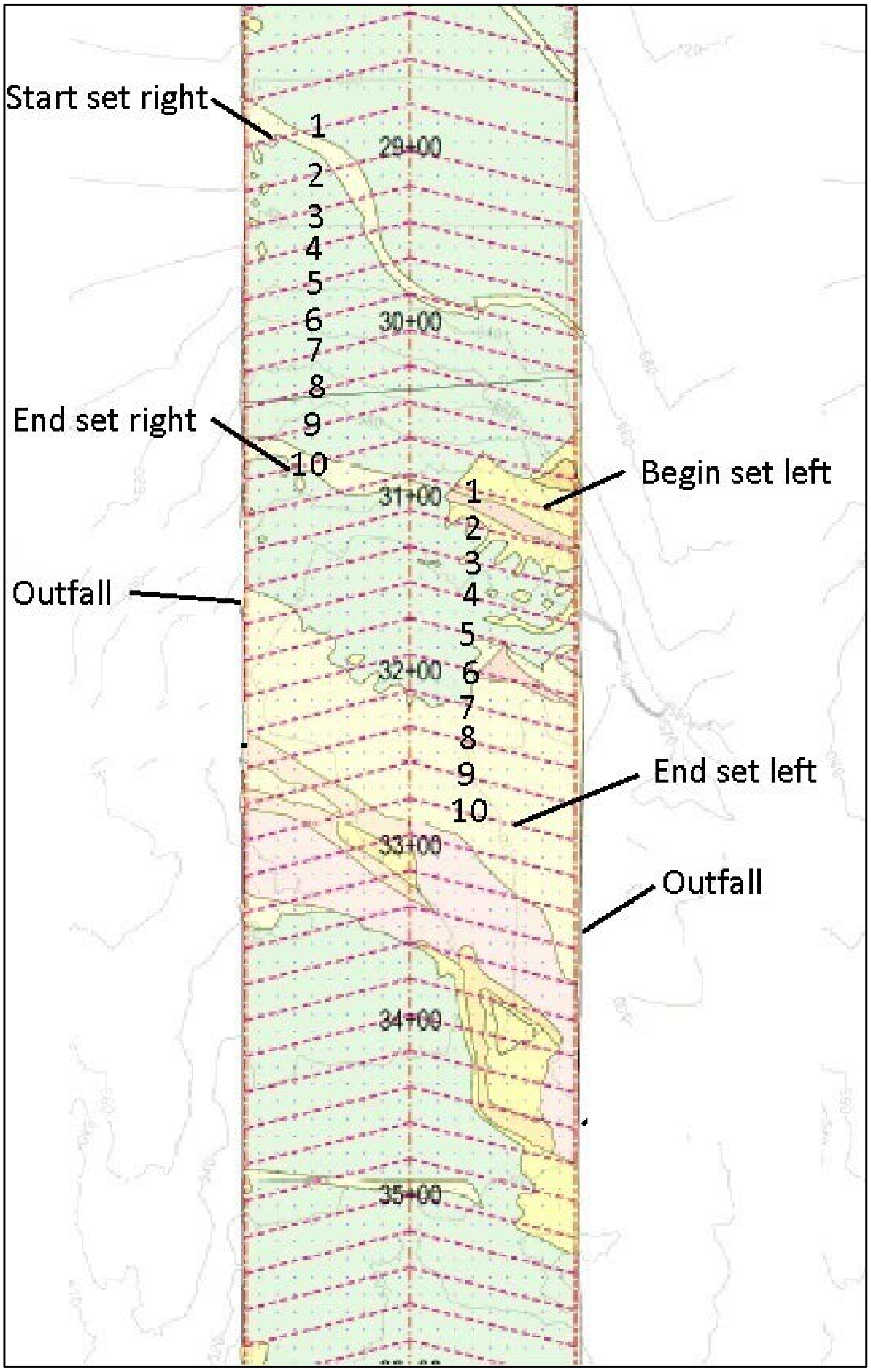


**Figure I-21: Problematic Drain Sets (from DWR)**

Only one construction photo from the foundation areas between Sta. 33+00 and 35+00 could be found in the documentation provided to the IFT (see Figure I-11). It is possible that no other photos exist in the official records of Oroville construction. However, as discussed in Appendix A, foundations in other moderately to strongly weathered foundation areas, as seen in construction photos, show a considerable amount of soil-like material that is left on the surface after cleanup. Deep holes in this area as seen in the post-failure photos likely developed after the chute slabs failed and the weak foundation materials were exposed to direct discharge from the spillway chute above. Prior to that, flows leaking into the foundation would not have enough energy to remove that much material, and the coarse materials such as rock fragments would be too large to pass through the drains.

One issue that has been the subject of much speculation since the failure occurred is the possibility that a deep void formed below the chute slab around Station 33+00. The IFT heard this comment from several interviewees. Certainly there was a considerable amount of erodible foundation material in that area. However, as stated above, this material likely contained rock fragments that would not likely be moved by low flows under the slab, and rock fragments could not pass through the gravel surrounding the drains or the drain perforations. Spillways founded on soil foundations, such as Reclamation's Hyrum Dam Spillway, have experienced considerable foundation erosion, with deep channels. In such cases, there was mostly fine-grained material in the foundation that has eroded. There was also evidence of this erosion before the chute slab failed, because it takes a long time and many spill events for large voids to form under the slab. The evidence of erosion includes cracking of the concrete as it settles into the voids, and offset cracks and joints. At Oroville, there appears to be fairly shallow rock below the surface at the downstream end of the initial failure area. If a large void had formed between Station 33+00 and 34+00, the slab could eventually sag as the void grew and the underside of the slab was unsupported by foundation material. This would be evident by offsets at joints or cracks between the area where the slab foundation eroded and downstream, where rock prevented erosion. This condition was never noted or observed from photographs of this area. Additionally, even if anchor bars were only partially embedded in good rock, they act as struts that could prevent the slabs from sagging into a shallow void, if one formed beneath the slab. There was also no unusual cracking observed in this area, which would have indicated settlement. Therefore, the IFT has concluded that, if erosion had occurred in this area, it was not likely extensive enough for the slab to collapse before the foundation was exposed to direct flow from the chute.

## 6.2 Drainage Capacity

The capacity of the collector drain is determined by free-surface flow conditions in the portion between the collectors and the outfall, see Figure I-22.

This 12-inch VCP outfall pipe is set on a 0.005 slope. If the pipe becomes 82 percent full, it can carry the same amount of water as if it were full. With a Manning Coefficient n = 0.012, the maximum discharge when the discharge collector pipe flows full is 3.25 cfs. For higher flow rates, the collector pipe and herringbone drains will flow full. In Appendix B, the computed flow into a crack in the damaged area through a ½-inch gap is given as 0.28 cfs/foot when the spillway discharge is 54,000 cfs. At this rate, the flow through just 16.75 feet of open transverse joint matching this condition would be 4.7 cfs. Thus, flow through transverse cracks or joints, if only a small percentage of the cracks or joints were partially open, could exceed the capacity of the collector drains. With over 800 feet of herringbone drains feeding a single collector drain, the cumulative flow from all contraction joint and herringbone crack repair failures within the collector drain set would add to the total discharge. The local drain set flows combined with possible flow passing under the slab from upstream of Station 33+00 when the failure occurred, could have easily exceeded the collector drain capacity, allowing pressure to back-up under the slab.

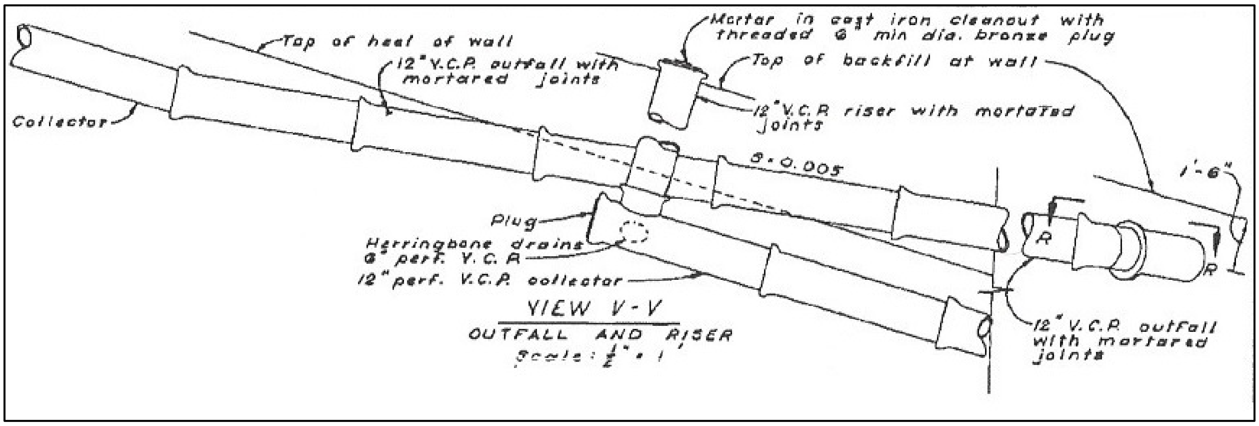


**Figure I-22: Collector Drain Detail [I-7]**

This 12-inch VCP outfall pipe is set on a 0.005 slope. If the pipe becomes 82 percent full, it can carry the same amount of water as if it were full. With a Manning Coefficient n = 0.012, the maximum discharge when the discharge collector pipe flows full is 3.25 cfs. For higher flow rates, the collector pipe and herringbone drains will flow full. In Appendix B, the computed flow into a crack in the damaged area through a ½-inch gap is given as 0.28 cfs/foot when the spillway discharge is 54,000 cfs. At this rate, the flow through just 16.75 feet of open transverse joint matching this condition would be 4.7 cfs. Thus, flow through transverse cracks or joints, if only a small percentage of the cracks or joints were partially open, could exceed the capacity of the collector drains. With over 800 feet of herringbone drains feeding a single collector drain, the cumulative flow from all contraction joint and herringbone crack repair failures within the collector drain set would add to the total discharge. The local drain set flows combined with possible flow passing under the slab from upstream of Station 33+00 when the failure occurred, could have easily exceeded the collector drain capacity, allowing pressure to back-up under the slab.

The computations for flow into cracks and open joints as presented above and in Appendix B are assuming only minor offsets into the flow. Where spalling occurred above a crack or joint, the offset into the flow at the deepest part of the spall could be several inches, which would result in much higher stagnation pressures and flow into the foundation. If the top of a herringbone pipe and surrounding gravel envelope were to be exposed to chute flows, foundation inflows could be significantly higher. In addition to the potential spall at Station 33+00 and at nearby cracks and joints, the downstream slab may have slipped downstream along the weak foundation material left on the surface before the slab was placed. This movement could open a gap that would allow water to enter the foundation more easily. The tendency of the slabs to slide downstream can be seen in the gap that developed in the existing spillway at Station 29+00 following the failure of the chute up to Station 30+00, Figure I-23. In the winter months, the slab joint and crack can be expected to be near their maximum opening. If a slab has a tendency to slid downstream, it would likely happen during the winter with flows being injected into the foundation.


**Figure I-23: Gap in Invert Slabs at Station 29+00 [I-2]**

## 6.3 Concrete Spalling and Delamination

Many of the repairs over the life of the spillway structure were made to repair concrete spalling and delamination at cracks and joints. These repairs were discussed above and in Appendix G. The southwest facing spillway chute surface may heat up considerably during summer months when the solar heat gain is added to the already high ambient air temperatures. The heated concrete surface may expand considerably. The resulting high compression stresses at the surface, as well as the expected temperature differential between the concrete surface and the bottom of the slab can produce high temperature stresses within the concrete. Combining these stresses with stresses produced by exposed reinforcing bars or dowels that expand due to corrosion at the face of a crack or joint, it is no surprise that spalling and delamination has occurred at some of these features. Unless these stresses are relieved, even the best repair effort is likely to fail during subsequent seasonal temperature cycles. The spillway has a history of past concrete repairs failing. It is likely that high velocity spillway discharges, such as those experienced in February 2017, expedite these failures.

A forensic investigation of the existing spillway was conducted by cutting sections of the chute invert away and lifting out sections of the concrete (see Appendix D). This investigation showed that many of the repairs had delaminated from the original concrete and that in some cases, the delamination extended into the previously undamaged slab, Figures I-24 and I-25.


**Figure I-24: Delamination of Slab (IFT photo)**


**Figure 25: Delamination of Slab (IFT photo)**

## 6.4 Cracks and Damage Areas in Failure Zone

Cracks in the slabs were documented for the 2009 repairs of the chute [I-5], see Figure I-26. The number next to the crack indicates the length of the crack. The number next to the hatched area indicates the square feet of the spall repair and the number next to the shaded area indicates the square feet of the surface repair. It can be seen from the figure that almost every crack required repair. Given this, whatever repairs that were made to cracks in this area in the past did not hold up to the past temperature cycles and chute flows. Reports from the 2009 repairs indicate that the cracks on the right side of centerline were filled with non-shrink grout (see Appendix G). It is likely that this type of grout could have failed in compression and was washed out with subsequent flows. The left side was filled with a flexible material that may have been able to survive some gap closure, but it is likely with cracks that are caused by low temperature conditions, that they will completely close during high temperature periods (summer months). It is not reasonable to expect any joint filler material to survive these joint closures without being squeezed out of the joint or failing. It is therefore concluded that joint filler could lose its effectiveness after just one annual temperature cycle. Even if the flexible filler material is able to rebound after one summer season, by 2017 these cracks had experienced three summer cycles since the last repair.

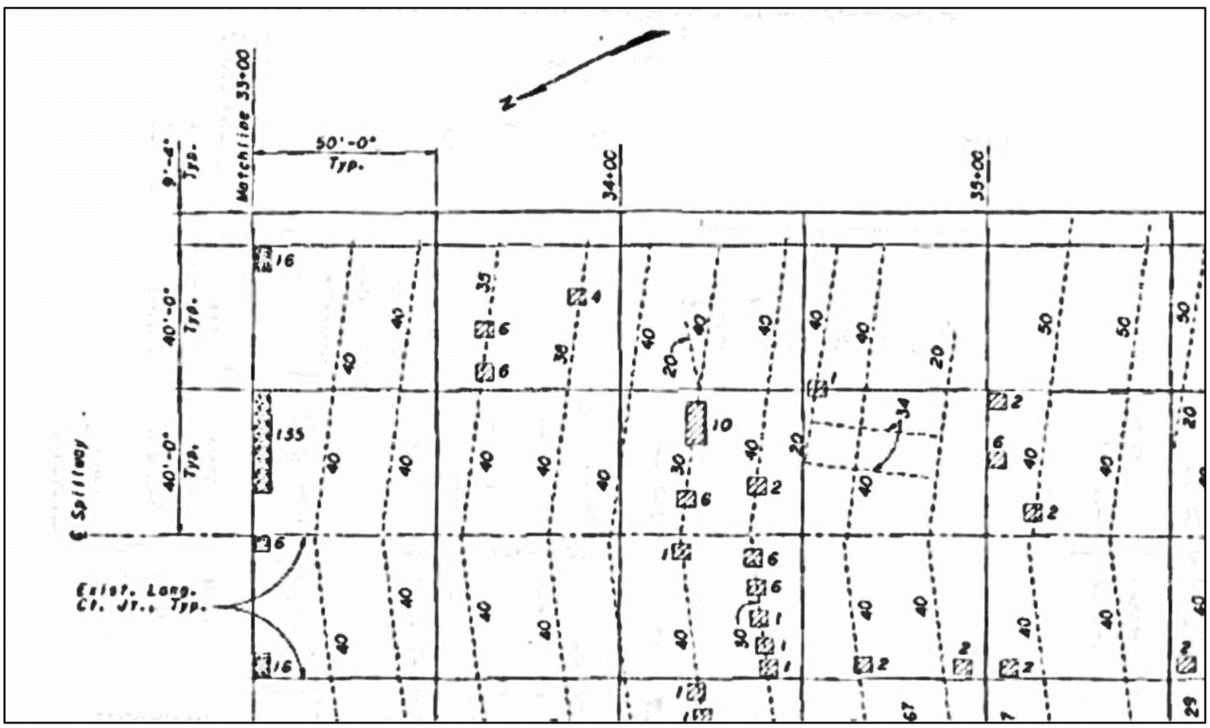


**Figure I-26: Cracks in Invert of Chute in 2009 [I-5]**

## 6.5 Observations in Damage Area

Figure I-27 shows that the slabs in Lane 5 downstream of Station 34+50 were removed to approximate Station 35+50. A detail view of the downstream end of this area is shown in Figure I-28. The slabs in Lane 4 all have damage to the longitudinal key and adjacent concrete. In comparison the slabs upstream of Station 35+50 in Lane 6 do not show similar damage, although there are significant spalls between Station 35+00 and 35+50. The observed conditions in Lane 6 are the result of the key at the Lane 5-6 being configured such that the Lane 5 concrete laps over the Lane 6 concrete. Consequently, the spalling is caused is caused by pull out of the dowels only, and the key does not need to be sheared for the Lane 5 slab sections to be lifted out. In contrast, at the Lane 4-5 joint the lapping key concrete in Lane 4 must be sheared for the Lane 5 slab sections to lift out. In combination this observed damage indicates that the slabs in Lane 5 between Stations 34+50 and 35+50 were likely rotated out of position about an axis that corresponds with the downstream end of the damage in Lane 5. Based on the evidence from these images, it seems that this is a location where the slab was jacked out as high velocity spillway chute flows produce high stagnation pressures beneath the slab. The edge spalling seen in figure I-28 and the bent bars seen in Figure I-10 are considered signs that the slab failed by jacking.

The downstream end of the slab at Station 33+00 does not show the fracture pattern at the key that would have been necessary if the slab at that location jacked, lifting the upstream edge of the slab into the flow. As discussed above, offsets into the flow associated with an open crack or joint are necessary to generate the kinds of stagnation pressures that can lift a slab. The lack of spalling indicates that the slab immediately downstream of Station 33+00 slid downstream or fell into the erosion hole before being removed. The longitudinal slab joint in Lane 2 just downstream of Station 33+00 also does not show any fractures from dowels pulling out like the downstream end of the Lane 5-6 joint. The key was formed such that the Lane 3 slab that was removed overlapped the Lane 2 slab that remains. Still, the lack of damage on this and the upstream edge is further indication that the slab immediately downstream of Station 33+00 did not jack out in an upward movement. This seems to present evidence that the failure occurred downstream from Station 33+00, opening a hole for the slab at Station 33+00 to slide or fall into.

Downstream from about Station 33+50 on the right side of the failure, a portion of the Lane 2 slab broke off during the failure. This can be seen in Figure I-29. In this still image captured from a drone video from February 8, 2017, spalling similar to the spalling in Figure I-28 can be seen along the edge of the failure. The IFT reviewed many drone video files from that day, and it can be seen from those videos that all of the broken edges of concrete where the slab failed downstream of approximate Station 33+50 (also seen in Figure I-10), show spalling of the remaining concrete on the top surface, and rebar that is bent downstream from the edge of the slab break, which tends to indicate the slabs rotated downstream as the upstream edge was jacked upward into the flow. This provides strong indications that the initial failure occured upstream of approximately Station 33+50.


**Figure I-27: Detail of Initial Spillway Damage [I-2]**


**Figure I-28: Detail of Damage Downstream of Station 34+50 [I-2]**

**Figure I-29: Right Side of Failure Area Looking Upstream – Captured from Drone Video [I-2]**

## 7.0 POSSIBLE FAILURE SCENARIO

By the time any of the eye witnesses were able to observe and document the chute failure with videos and photos, the failure was already in progress. There were no visual observations that could identify the precise location of the failure. By the time the chute flows were shut down, additional areas of the chute slab had failed, and the physical evidence had been washed downstream. Therefore, it will not be possible to identify the exact cause and location of the failure.

### 7.1 Chute Slab Failure Factors

Although the exact initial failure location cannot be identified with precision, the critical physical factors that led to the failure can be stated with some degree of confidence. The following factors are believed to be the most likely factors contributing to the initial chute slab failure on February 7, 2017:

A. Water entered the foundation beneath the slab through:

1. Cracks in the slab above herringbone drains or elsewhere. An offset into the flow may have been present at a crack for any one of the following reasons:
    a. A local spall on the upstream side creating an offset into the flow
    b. Minor foundation erosion caused a slab section to sag, creating offsets at the cracks
    c. Elastomeric joint filler detached from the upstream side leaving a flap on the downstream side that projected into the flow

2. Open joints
    a. Sawcut joints lost filler material and the resulting gap allowed inflow into the joint
    b. Upstream spalling at contraction joints that resulted in a vertical downstream face
3. Holes in the slab such as a spall above the drains that connects to the foundation through the gravel drain material, because of:
    a. Design of the herringbone drains that created weak areas in the slabs
    b. Repairs that created delamination and weakly bonded surfaces within the slab over time from exposure to hot and cold cycles

B. Flows into the foundation exceeded drainage capacity due to:
1. Change in spillway discharge from 42,500 cfs to 52,500 cfs
2. Collector drains were full, which caused a backup into the herringbone drains, restricting flow into the herringbone drains
3. Fines and small rock fragments partially plugged the gravel envelope surrounding the herringbone drains, restricting drain inflows
4. Foundation inflows were higher than individual drains could pass
5. The good foundation with good anchorage and bonding to the rock at the downstream end of the damaged area (possibly around Station 35+00 to 35+50) prevented passage of flow under the slab at that point, forcing inflows to exit through a limited number of herringbone drains upstream
6. Lack of waterstops at joints
7. Lack of cutoff walls at the downstream ends of the slabs in the weak shear zone resulting in under slab flows from the upstream drainage set to flow to the downstream drainage set where the failure occured

C. Resulting pressures under the slabs exceeded the uplift resistance from the following resisting forces:
1. Weight of the slab
2. Weight of water above the slab
3. Rock to concrete bond strength (likely to be low to none at this location in the foundation)
4. Anchor bar strength
    a. Foundation may have been too weak to fully develop the anchor bar strength at this location
    b. Corrosion may have reduced anchor bar strength
    c. Anchor bar grouting may have been ineffective due to open joints or voids in the foundation that resulted in grout loss

5. Slab bending or shear strength was exceeded
    a. Failure of an anchor bar doubled the effective span
    b. Pre-existing cracks
    c. Failed reinforcing bars

Once the uplift resistance was compromised, the slab section lifted into the flow. This would expose a higher vertical face to the high velocity chute flow, where more flow and pressure could develop and pass into the foundation. The more the slab lifts, the more pressure is generated. Once the local slab section becomes unstable it jacks out in a sudden, explosive failure. This would be followed by considerable spray as the failed slab is pushed downstream and the chute flow drops into the void where the slab was removed. This seems to be what was described by the eyewitnesses on site when the failure initiated. Altough they did not see the slab fail, they heard it and saw the spray.

From the factors above, a possible failure scenario can be suggested. Prior to failure on February 7, 2017, the chute slab was compromised by failed repairs or new damage. Flows into the foundation had increased as a result. Local herringbone drains were at or near capacity, and pressure under the slab was possibly already increasing. As the flow was increased, either a sudden repair or spall failure occured at or downstream from Station 33+00, and created an additional location for foundation inflow and critical stagnation pressure to develop; or the increased flow itself was enough to produce higher foundation inflow and stagnation pressure. This resulted in significant additional flow entering the foundation and exiting through the herringbone drains. As the drainage capacity was exceeded, pressure built up under the slab, reaching a critical point in terms of slab stability. As the spillway flow increased from about 42,500 cfs to 52,500 cfs, the stagnation pressure increased by approximately 13 percent. The increase in stagnation pressure from a spillway flow increasing from 48,000 cfs to 54,500 cfs may have been on the order of 10 percent.

Additional water may have also been flowing from the drainage set starting at Station 31+00 and joined the flow entering the foundation near Station 33+00. The flow passed under the slab until it reached the hard rock foundation at the downstream end of the damaged area. At this point, the flow could no longer freely pass under the slab, but had to enter the drainage set starting at Station 33+00. If water was coming from upstream of Station 33+00, and was exceeding the capacity of the downstream drains, it could have eventually backed up to the upstream drain set, but to do this, the pressure would need to build around the downstream drains. As the spillway flows increased, the stagnation pressure from the flowing water could have suddenly increased to as much as 70 feet of head. This pressure at the underside of the slab could create enough force to jack up a portion of the slab, even if the anchor bars were at full strength. Since the failure did not occur anywhere else in the chute where the foundation was better, it is more likely that the pressures were not this high, but were high enough to fail a weakened slab section with low anchor bar pull-out strength.

The slabs downstream to about 35+50 were then lifted and removed by the flowing water. This probably occurred after the initial slab section failed. Note that a portion of the slab in Lane 4 downstream of Station 35+00 was lifted and partially broken, see Figure I-28. This damage in Lane 4 likely occurred as the slab in Lane 5 was lifted and load was transferred to Lane 4 through the keys and dowels at the joint.

Simultaneously, slabs upstream of initial failure were likely undermined and removed once foundation support was lost and the joints or cracks opened enough to expose the underside to direct flow. The process of head cutting then proceeded laterally and upstream to Station 33+00, leaving the hole seen in Figures I-1, 1-9 and I-27.

Videos, photographs, and observations of the DWR personnel on site at the time of the initial damage seem to validate this scenario. For example, the slab downstream from Sta. 33+00 seems to be intact in earlier videos but was gone in later videos.

## 8.0 SUMMARY

Although the exact initial failure location cannot be identified with precision, the critical physical elements that led to the failure can be stated with some degree of confidence. These are as follows:

- The weak and erodible foundation between Stations 31+00 and 35+00 that allowed piping (internal erosion) under the foundation to occur and reduced anchor capacity,
- The good foundation downstream of the initial failure area that provided a downstream limit to the piping (internal erosion) and allowed pressures to build up under the slab,
- Good anchorage in the more competent downstream rock,
- Failure of previous spillway repairs at the joints and the cracks over the herringbone drains, which allowed increased seepage into the foundation over the entire chute,
- The possible repair failure or spall over a drain near Station 33+00 that contributed to the sudden increase in flow and pressure that led to the loss of portions of the slab downstream,
- Lack of water stops in the slab joints, which allowed inflow to the foundation,
- Design of the herringbone drains that created weak areas in the slabs and promoted cracking,
- Repairs that created delamination within the slab and under the repairs with time due to improper placement and exposure to hot and cold cycles, and
- Lack of cutoff walls at the downstream ends of the slabs in the weak shear zone.

## 9.0 REFERENCES

[I-1] Google Earth Photograph that has been modified.

[I-2] DWR Staff Photos and Videos.

[I-3] Hiker’s photo. See also YouTube.com Lake Oroville dam spillway disaster. First onsite!

[I-4] Final Geologic Report, Oroville Dam Spillway, State Water Facilities, Oroville Division, Butte County California, Appendix A to Final Construction Report, Contract No. 345284, Specifications No. 65-09, March 30, 1970.

[I-5] Department of Water Resources, Division of Engineering, State Water Facilities, Oroville Field Division, Oroville Dam, Antalope Dam, Frenchman Dam, and Grizzly Valley Dam, Spillway Repairs, Specifications No. 09-14, 2009.

[I-6] Mercury News. http://www.mercurynews.com/2017/03/11/oroville-dam-photos-taken-weeks-before-spillway-broke-show-something-wrong/ and http://photos.orovillemr.com/2017/01/31/photos-oroville-week-of-1-30-2017/#2

[I-7] DWR Drawing A-3B9-1, Sheet 47, Feb 26, 1966, Spec No. 65-09.

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# Appendix J

# Human Factors Framework and Methodology

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**TABLE OF CONTENTS**

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This appendix provides general background on what the term “human factors” involves, describes the human factors framework which was applied during the IFT’s investigation of the February 2017 Oroville spillway incident, describes the methodology the IFT used for investigating human factors aspects of the incident, and highlights some higher-level human factors contributing to the incident.

## 1.0 BACKGROUND ON HUMAN FACTORS

The field of “human factors” considers how and why systems meet or do not meet performance expectations, with an emphasis on understanding and prevention of incidents and failures[1]. The systems considered in human factors work typically include both human and physical aspects, and are sometimes referred to as “sociotechnical” systems.

The range of human factors which may be considered spans scales of individuals, groups, organizations, industries, and the broader social, economic, and political context. Accordingly, human factors involves application of social science and draws on knowledge from fields such as psychology, social psychology, sociology, cultural anthropology, management, economics, political science, and history. At the same time, because human factors approaches are often applied to physical systems, such as dams, specialist knowledge of these physical systems is also necessary. As a result, human factors is a highly interdisciplinary field.

The field of human factors has evolved and grown during the past few decades, and a variety of frameworks have been developed. These frameworks generally have overlapping aspects, but with some variety in their assumptions and the aspects they emphasize. Therefore, each framework has particular strengths and limitations. The literature on human factors is very extensive, and references [J-1] through [J-25] are a selected sample of the literature, which describe many human factors frameworks.

Human factors approaches have been extensively applied in fields such as aviation, nuclear power, chemical processing, and health care. The application of human factors approaches specifically to civil infrastructure, particularly dam safety, is more recent, with most of this work having occurred during the past decade. References [J-26] through [J-45] describe some of this work, with an emphasis on applications to dams.

## 2.0 HUMAN FACTORS FRAMEWORK

The human factors framework used for this IFT investigation represents a synthesis of various perspectives. It was previously developed by one of the IFT team members (Alvi) specifically for dam safety and has been refined during the past decade based on extensive literature review, review of dozens of case studies of past incidents and failures of dams and other systems, and application to several prior dam failure investigations [J-35, J-39, J-40, J-41, J-42, and J-45]. This Section 2 describes this framework.

[1] Reference to “failures” in this appendix is intended to include major incidents.

## 2.1 Key Observations and Assumptions

The human factors framework used for this IFT investigation is based on the following observations regarding past failures of dams and other systems:

- Failures are typically preceded by interactions of physical and human factors which begin years or decades prior to the failure [J-7].
- The interactions among physical and human factors are often not simple and linear. Instead, they may be complex and involve nonlinear relationships, feedback loops, causes having multiple effects, effects having multiple causes, and a lack of distinct "root causes" or dominant contributing factors [J-5, J-9, J-14, J-16, and J-20].
- Interactions among physical and human factors usually generate "warning signs" which are not recognized, or not sufficiently acted upon, prior to failures [J-25].
- Physical processes deterministically follow physical laws, with no possibility of physical "mistakes." Therefore, failures – in the sense of human intentions not being fulfilled – are fundamentally due to human factors, as a result of human efforts individually and collectively "falling short" in various ways. A story of *why* a failure happened therefore cannot be complete without reference to contributing human factors [J-7].
- A natural tendency is for systems to move towards disorder and failure, in line with the concept of increasing entropy[2] in physics. Therefore, systems such as dams are typically not inherently "safe" [J-6], and continual human effort is needed to maintain order and prevent failure [J-3, J-15, and J-21].
- Systems such as dams, including the people involved in designing, building, operating, and managing them, tend to conservatively have numerous "barriers" which must be overcome for failures to occur [J-3 and J-11]. This generally makes failures unlikely and results in very low overall failure rates. However, when dealing with a large number of systems, such as the approximately 90,000 dams in the United States, it can be expected that "unlikely" failures will sometimes occur, due to physical and human factors "lining up" in an adverse way that overcomes all barriers [J-3].

With these observations in mind, the propensity towards failure can be viewed as being determined by the balance of human factors which contribute to failure ("demand") versus those which contribute to safety ("capacity"). Thus, applying a standard engineering metaphor, failure results when human factors demand on the system exceeds capacity, and safety results when capacity exceeds demand.

## 2.2 Primary Drivers of Failure

The human factors contributing to safety "demand," and therefore the potential for failure, can generally be placed into three categories of *primary drivers* of failure:

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[2] Entropy is a quantitative measure of the extent of disorder of a system.

- **Pressure from non-safety goals** [J-20] such as delivering water, generating power, reducing cost, meeting schedules, building and maintaining relationships, personal goals, and political goals.
- **Human fallibility and limitations** associated with misperception, faulty memory, ambiguity and vagueness in use of language, incompleteness of information, lack of knowledge, lack of expertise, unreliability of intuition, inaccuracy of models [J-46], cognitive biases operating subconsciously at the individual level [J-47 through J-50] and group level [J-50 and J-51], use of heuristic shortcuts [J-48], emotions, and fatigue.
- **Complexity** resulting from multiple system components having interactions which may involve nonlinearities, feedback loops, network effects, etc. Such interactions can result in large effects from small causes, including "tipping points" when thresholds are reached, and they make complex systems difficult to model, predict, and control [J-5, J-20, and J-52]. Complexity generally exacerbates the effects of human fallibility and limitations.

## 2.3 Human Error

The primary drivers of failure lead to various types of "human errors," which can include categories such as "slips" (actions committed inadvertently), "lapses" (inadvertent inactions), and "mistakes" (intended actions with unintended outcomes, due to errors in thinking) [J-1 and J-2]. In the context of dam safety, mistakes are the most common type of human errors that contribute to failures. "Violations" are also sometimes classified as a category of human errors, and involve situations in which there is deliberate non-compliance with rules and procedures, usually because the rules or procedures are viewed as unworkable in practice [J-1].

In general, with all categories of human errors, judgments regarding what constitutes "error" are usually made in retrospect, and are subject to the pitfalls of hindsight bias[3] and fundamental attribution bias[4] [J-14]. Care must therefore be taken in forensic investigations to avoid readily assigning "blame" [J-53]. Instead, investigators must put themselves in the shoes of the people whose decisions and actions are being evaluated, recognizing that they faced pressures from their situational contexts, were inherently fallible and limited, and did not have the benefit of clear foresight when they made their decisions and took their actions [J-14]. Moreover, it must be recognized that factors beyond the control of an individual or group can sometimes result in generally "good" decisions and actions having undesirable outcomes, and generally "bad" decisions and actions sometimes having desirable outcomes; the role of "luck" cannot be entirely eliminated [J-54].

Based on these considerations, identification of "human errors" is not a sufficient endpoint for a forensic investigation, and assigning blame to individuals is often unreasonable and counterproductive [J-14]. Instead, identified human errors should be treated as prompts to

[3] Hindsight bias is the tendency to overestimate how predictable an event was, after the event has already occurred.

[4] Fundamental attribution bias is the tendency to attribute undesirable outcomes of others to them personally, while attributing our own undesirable outcomes to situational factors.

investigate further and delve deeper, to understand the situational and contextual factors which contributed to those human errors [J-10, J-13, J-14, and J-17]. That is the approach the IFT endeavored to take for this investigation.

## 2.4 Risk Management

With the above caveats regarding "human errors" in mind, human errors and the underlying primary drivers of failure noted in Section 2.2 often lead to inadequate risk management. There are three specific types of inadequacy in risk management due to human errors:

- **Ignorance** involves being insufficiently aware of risks. This may be due to aspects of human fallibility and limitations such as lack of information, inaccurate information, lack of knowledge and expertise, and unreliable intuition. Complexity can also contribute to ignorance.
- **Complacency** involves being sufficiently aware of risks, but being overly risk tolerant. This may be due to aspects of human fallibility and limitations such as fatigue, emotions, indifference, and optimism bias ("it won't happen to me"). Pressure from non-safety goals can also contribute to complacency.
- **Overconfidence** involves being sufficiently aware of risks, but overestimating ability to deal with them. This may be due to aspects of human fallibility and limitations such as inherent overconfidence bias, which results in overestimating knowledge, capabilities, and performance [J-48 to J-50].

## 2.5 Safety Culture

Counterbalancing the drivers of failure described in Sections 2.2 through 2.4, the human factors contributing to system capacity for safety generally emanate from what is referred to as "safety culture" [J-8]. While this term is sometimes interpreted as applying specifically to worker safety and prevention of injuries on the job, the concept of safety culture is much more general and refers to safety of any system, including dams.

The general idea of safety culture is that individuals at all levels of an organization place high value on safety, which leads to a humble and vigilant attitude with respect to preventing failure [J-25]. For such a safety culture to be developed and maintained in an organization, the senior leadership of the organization must visibly give priority to safety, including allocating the resources and accepting the tradeoffs needed to achieve safety.

## 2.6 Best Practices

Experience in dam safety shows that strong safety cultures naturally lead to implementation of numerous "best practices" for dam safety risk management, with the understanding that these best practices need to be continually challenged, and, therefore, they evolve as the industry learns and improves. As a corollary, dam incidents and failures are typically preceded by long-term cumulative *neglect* of numerous accepted best practices. These best practices can be organized into two categories: general design and construction features of dam projects, and general organizational and professional practices.

Best practices for general design and construction features of dam projects include the following:

- **Specific design and construction best practices:** Generally-accepted best practices for specific aspects of design and construction should be identified and applied.
- **Design conservatism:** Designs should be sufficiently conservative and provide factors of safety commensurate with uncertainties and risks. To the extent possible, designs should also preferably provide physical redundancy, robustness, and resilience, as well as failure modes which generate warning signs.
- **Design customization:** Designs should be customized to suit features of project sites. This involves "scenario planning" during design to be ready to handle situations which may potentially be encountered during construction, testing during construction to verify that design assumptions and intent are met, and design adaptation during construction to address observed conditions.
- **Budget and schedule contingencies:** Provisions should be made for accommodating reasonable contingencies when establishing design and construction budgets and schedules.

Best practices for general organizational and professional practices, which encompass all project phases and tasks, include the following:

- **Resources and resilience:** Sufficient budget and staffing resources should be provided, so that systems and people are not stretched to their limits, thereby increasing error and failure rates [J-20]. The organization should also be resilient, in the sense of having sufficient internal diversity and adaptive capability to provide a broad and flexible repertoire of possible responses to cope with the potential challenges faced by the organization [J-12].
- **Humility, learning, and expertise:** Individuals and organizations should humbly recognize the limitations of their knowledge and skills, engage in continuing education and training, learn from study of past incidents and failures, and collaboratively draw on expertise, wherever it may be found, rather than simply deferring to authority based on position in a hierarchy [J-25].
- **Cognitive diversity:** Teams should have cognitively diverse membership, to bring in diversity of perspectives, education, training, experience, information, knowledge, models, skills, problem-solving methods, and heuristics [J-51 and J-55]. With effective team leadership, structure, and group dynamics, cognitively diverse teams can avoid problems such as groupthink[5] (see Appendix F3), and can outperform more homogeneous teams of the "best" people.
- **Decision-making authority:** Decision-making authority should be commensurate with responsibilities and expertise, rather than this authority being contravened by

[5] Groupthink is a phenomenon in which deliberation, judgment, and decision-making of a group and its members are compromised due to the social tendency of group members to seek harmony and coherence.

organizational structure [J-25]. This is particularly the case for safety personnel, who should be selected for their positions based on having relevant experience, vigilance, caution, humility, inquisitiveness, skepticism, discipline, meticulousness, communication ability, and assertiveness.

- **System modeling:** Appropriate system models should be developed, with a full range of potential failure modes identified, and emergency action plans developed accordingly. For actively operated systems, such as large hydropower dams, these failure modes should include operational failure modes, and it may be appropriate to explicitly account for interactions of physical and human factors in the system models. Where models are implemented through software, the software should be carefully developed, validated, and used [J-17].
- **Checklists:** Checklists should be used to reduce the incidence of human errors, especially for tasks which are relatively recurrent, such as inspections [J-56]. Checklists should be customized for each situation, clear and unambiguous, focused on items which are important but prone to being missed, prepared at a level of detail appropriate for the time available to use the checklist, and regularly updated based on experience. Recognizing that checklists are most effective for prevention of slips, lapses, and violations, but somewhat less effective for prevention of mistakes (see Section 2.3), checklists should be used to supplement, not replace, situation-specific attentive observation and critical thinking.
- **Information management:** Information management should involve thorough, well organized, and readily accessed documentation; open and collaborative information sharing within and across organizations; and not being dismissive of dissenting voices. This will enable surfacing and synthesis of fragmentary information to help "connect the dots" and better understand system behavior [J-23 and J-25].
- **Warning signs:** There should be vigilant monitoring to detect "warning signs" that a system is headed towards failure, while there is still a "window of recovery" available [J-25]. This monitoring should be conducted at regular intervals, after unusual events, and also during apparent "quiet periods." Once potential warning signs are detected, there should be prompt and appropriate investigative follow up, verification of that follow up, thorough documentation of observations and findings so that emerging patterns can be discerned and evaluated, and prompt implementation of any needed remedial actions. As a heuristic to help judge whether a potential warning sign warrants action, "simulated hindsight" can be used: fast-forward into the future, imagine that failure has occurred, and ask whether ignoring the potential warning was justifiable; if not, take the potential warning sign seriously.
- **Standards:** High professional, ethical, legal, and regulatory standards should be maintained – especially when lives are at stake.

In summary, organizations which have the capacity to handle demands on safety from various drivers of failure have a strong safety culture and diligently implement numerous best practices. Such organizations are mindful, cautious, humble, oriented towards learning and improving, resiliently adaptive, and maintain high professional and ethical standards. They vigilantly search for and promptly address warning signs before problems grow too large, and they make effective use of available information, expertise, resources, and management tools to properly balance safety against other organizational goals.

## 3.0 HUMAN FACTORS METHODOLOGY

The scope of human factors which the IFT considered in its investigation included judgments, decisions, actions, inactions, influencing situational factors, and interactions of the following parties: individuals in DWR, DSOD, and FERC; divisions, branches, and sections within DWR; DWR, DSOD, and FERC at the organizational level; DWR's external consultants; and the broader dam engineering and safety community and industry in the United States.

In-depth investigation of DSOD, FERC, or their dam safety programs was not considered to be within the scope of the IFT's investigation. However, the role of these regulators and the associated regulatory framework was considered with respect to contributing factors to the February 2017 incident.

With regard to these various parties, the IFT applied the human factors framework described in Section 2 to formulate questions such as the following, most of which can be followed with the question "If not, why not?":

- Was the spillway design and construction sufficiently conservative, customized to the site, and based on generally-accepted best practices of the era for design and construction?
- Did the parties involved in spillway design, construction, operations, inspection, maintenance, repairs, five-year reviews, and PFMAs have sufficient technical expertise for the tasks they undertook? Did they recognize the limitations of their expertise? Were there any warning signs of the spillway failure and, if so, why were they missed?
- How much priority was given, both on paper and in practice, to dam safety versus non-safety goals such as delivering water, generating power, controlling costs, and meeting schedules? Did DWR generally have sufficient budgets and staffing to support the goal of dam safety? Did the individuals with key roles related to dam safety, such as the DWR Chief Dam Safety Engineer, have decision-making authority commensurate with their responsibilities?
- Were there any noteworthy cognitive biases, such as confirmation bias[6], which contributed to the incident?
- Did system complexity contribute to the spillway failure modes not being identified and prevented?
- Were checklists used to help with tasks such as spillway inspections?

[6] Confirmation bias is the tendency to selectively seek, interpret, recall, or dismiss new information in a way which confirms a preexisting belief rather than challenging it.

- Was DWR's information management for its dams and spillways adequate?
- How well did DWR groups involved with dam safety coordinate with each other? How well did DWR coordinate with its regulators and external consultants?
- Did DWR, its regulators, and its consultants meet professional, ethical, and legal standards?

Based on these types of questions, investigative hypotheses were progressively formulated, tested, and modified through an iterative process until, after gathering a relatively large amount of information, it was found that hypotheses aligned well with the full body of information. The information which was gathered was generally treated as confidential, with the understanding that it would not be shared outside the IFT. This information was gathered through a combination of the following processes:

- **Public request for information:** A public request for information, with an independent email box established to contact the IFT, was publicized through the media, ASDSO, and USSD. Numerous emails were received, and several individuals who contacted the IFT were interviewed.
- **DWR general request for information:** A general request for information was sent to all employees of DWR, with responses to be sent to the same independent email box as described above. Several emails were received, and some individuals who contacted the IFT were interviewed.
- **Document review:** DWR provided the IFT with access to a very large number of documents, totaling many thousands of pages. The IFT reviewed these documents extensively, which led to further rounds of document requests and reviews. While most of these documents were most relevant to physical factors, many were relevant to human factors also.
- **Interviews:** Interviews were conducted with individuals involved in various aspects of the spillways and the incident, or otherwise in a position to provide information relevant to the investigation. Individuals interviewed included current and retired DWR employees, DSOD employees, FERC employees, USACE personnel, and individuals associated with the original Oroville Dam design and construction. In total, more than 75 individuals were interviewed, either in person or by phone, and some individuals were interviewed more than once. Most interviews were in-depth and lasted more than an hour, in some cases much longer than an hour.

  In addition to interviewing individuals who reached out to the IFT, the IFT studied DWR organizational charts and other documents to identify the various divisions, branches, and sections in DWR which have relevance to dam safety. From these various DWR groups, the IFT selected individuals in both supervisory and staff positions to contact to request interviews, and nearly all contacted individuals agreed to be interviewed.

The IFT found that interviewees were generally very candid and thoughtful in their responses, and showed a sincere desire to help DWR, its regulators, the broader dam safety community, and the public.

- **Surveys:** Two surveys were conducted of DWR personnel. One survey focused on educational and training background, professional experience and registrations, areas of expertise, and involvement in dam and spillway engineering and safety. The other survey was more general and solicited opinions regarding DWR organizational culture, internal communications and coordination, utilization of consultants, dam safety priority, budgets, schedules, and contributing factors for the February 2017 incident. Participation in the surveys was voluntary, each survey was sent to about 200 people, and there were about 100 respondents to each survey.

In total, the IFT collectively spent thousands of hours on its investigation. A substantial portion of this time was focused on human factors aspects of the investigation, in which all six IFT team members participated. While investigation of human factors necessarily involves a degree of subjectivity associated with the backgrounds and perspectives which investigators bring to bear, the IFT notes that its team members are relatively diverse in these respects, and the team members were able to reach consensus regarding the findings presented in this report.

## 4.0 HUMAN FACTORS CONTRIBUTING TO OROVILLE SPILLWAY INCIDENT

The IFT's findings related to human factors contributing to the February 2017 incident are presented in detail in the main report and throughout most of its appendices. It is therefore not the intent of this Appendix J to present detailed IFT findings here. Instead, this Section 4 of Appendix J highlights some of the higher-level human factors contributing to the spillway incident, in terms of the human factors framework described in this appendix.

On the whole, the February 2017 spillway incident at Oroville Dam can be viewed as a "textbook" case of a major dam incident, with respect to typifying the general extent and types of human factors which contributed to the incident. The following are some of the higher-level contributing factors:

- Going all the way back to the design and construction of the spillways, the incident was preceded by decades of interactions and effects of human and physical factors, many of which were inadequate or detrimental, and the overall set of interactions was somewhat complex. During these decades, numerous warning signs of the spillway failure were missed, and many barriers, which were intended to provide "checks and balances," were overcome to eventually produce the spillway failure.

  Overcoming so many barriers can be thought of as involving a degree of "bad luck," but more importantly, it indicates a long-term *systemic* failure of DWR, regulatory, and general industry practices to recognize and address the deficiencies and warning signs that preceded the incident. The incident cannot reasonably be "blamed" mainly on any one individual, group, or organization.

- The inadequacies in dam safety risk management that contributed to the failure primarily involved ignorance about the existence of the risks associated with the spillways, which was mainly due to insufficient expertise regarding spillway failure modes and mischaracterization of the geology. Confirmation bias related to perpetuating misunderstanding of the geologic conditions at the spillways likely contributed to this ignorance of risks. There was also a degree of complacency regarding tolerating risks, and some overconfidence in ability to manage risks.
- DWR did not have a sufficiently strong dam safety culture, and dam safety did not have adequate priority relative to non-safety goals, such as delivering water, generating power, and controlling costs. DWR's Chief Dam Safety Engineer, who has been highly dedicated, was given major dam safety responsibility without commensurate decision-making authority and staff resources, and there was no identified DWR executive who the Chief Dam Safety Engineer reported to and who actively took primarily responsibility for understanding dam safety issues and balancing dam safety against other organizational goals. Instead, DWR had a problematic diffusion of responsibility[7] with respect to dam safety.
- Despite the spillways being major hydraulic structures, which were appurtenant to the tallest dam in the United States, the design and construction of the spillways fell short of the best practices of the era for a spillway of this size, and lacked conservatism and customization to the site. Lack of technical expertise contributed to these shortcomings, and placing adherence to budget and schedule over quality during construction may have also contributed.
- DWR, and to some extent also its regulators and external consultants, fell short in meeting most of the general organizational and professional best practices described in Section 2.6 of this appendix. Some key areas in which current best practices were not met include having sufficient qualified staff to meet workload demands, having sufficient technical expertise and humility about limitations in expertise, using suitable checklists to aid tasks such as inspections, having an information management system suitable for the very large amount of information associated with DWR's dams, having effective internal coordination and collaboration within DWR, and having a vigilant attitude towards detecting, investigating, and properly remediating causes of warning signs associated with the spillways.

Overall, in terms of human factors, the safety "demands" which contributed to failure were significant, while the systemic "capacity" to meet those demands and maintain dam safety was substantially lacking in many areas. Half a century after design and construction of the spillways, this systemic imbalance in human factors had set the stage physically for the spillway incident to occur in February 2017.

[7] Diffusion of responsibility is a phenomenon in which each individual in a group takes less responsibility for a decision or action because the group has multiple members and their individual responsibilities are not well defined.

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# Appendix K

# Dam Safety Organization and Practices

**K1 – Dam Safety Organization, Practices, and Culture Within DWR**

**K2 – General Regulatory Aspects**

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# Appendix K1

# Dam Safety Organization, Practice, and Culture Within DWR

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## 1.0 INTRODUCTION

A number of questions arose during the IFT's investigations where the general practice and culture of dam safety within DWR could have had influence in the timelines to failure. These include understanding why there was so little emphasis placed on issues such as:

- Questioning/reviewing the original designs and as-built conditions.
- Ensuring that the observations and investigations of the abnormally high service spillway chute underdrain flows were continued.
- Performing proper investigations into the erodibility of the foundation rock, especially in 2005, 2009, and 2014.
- Understanding the implications of the herringbone crack patterns in the service spillway chute.
- Whether those with direct dam safety responsibility and knowledge of the site had enough influence during the February 2017 emergency decisions that led to the use of the emergency spillway.

A distinction must be made between two of the branches of DWR: the Division of Operations and Maintenance (O&M) acting as dam owner, and the Division of Safety of Dams (DSOD) acting as regulator. This appendix deals with the O&M aspects; refer to Appendix K2 for regulatory aspects.

## 2.0 HISTORICAL PERSPECTIVE

There was immense pride in the achievements of the State Water Project in the 1960s. This pride is perhaps best captured by the following verbatim transcript of a meeting of the Executive Staff in December 1966 [K1-1]. By the end of 1966, five dams in the State Water Project had been completed, and five were under construction, including Oroville:

> "To achieve this outstanding record to date has required overcoming seemingly insurmountable obstacles in administration and engineering. Whereas Federal agencies for example, have had years of experience in large construction programs and have procedures tested and matured over the years, the department had to gain its maturity quite rapidly. A tremendous effort has been expended at all levels of management in adapting the unprecedented design and construction program to the State's procedures and, in turn, adjusting those procedures to accommodate the program.
>
> By 1961, the department's design staff had already reached a strength of 600. By late 1966, it had increased 40 percent to its current strength of 850. The construction force in 1961 totaled 160 employees. In 1966, it had increased over eight-fold to 1,320 employees. This staff, recruited from Federal and other agencies and many campuses, has grown quickly in maturity and stature and is generally recognized as one of the most outstanding engineering organizations in public service.

> At the close of 1966 … the behemoths of construction are beginning to move inexorably across the valleys and through the mountains of Southern California. History is the more meaningful to those who have played a part in its making. Yet, even for those who have contributed, and for those who have watched, there is a feeling of disbelief in looking back on our accomplishments. The annals of history of California will record what has been achieved on the State Water Project in the period 1961 through 1966. All recognize, however, that words alone are never adequate to record achievements or describe the stress of reaching unprecedented objectives. The best record of these achievements are the physical works we have constructed and will construct for the benefit of Californians for generations to come."

Through interviews, the IFT gained an impression that a strong sense of pride persists to the present day.

Clearly, the above suggests that one source of pride was that their accomplishments were separate from those of the Federal agencies (USACE and USBR). The IFT was able to interview a DWR retired employee from the time of construction, who reminisced that one executive "used to boast that DWR had as many geologists in the 60s as the Corps and Bureau combined." It is possible that cross-referencing other designs of the time would have been considered not necessary, as they believed that they had already assembled "one of the most outstanding engineering organizations in public service." [K1-1]

This pride has clearly persisted to the present day. The IFT gained an appreciation of the strong sense of pride among DWR's engineers through a number of different interviewees, many noting that DWR is a "proud, professional, competent organization" with employees having "DWR blue in their veins." A number of interviewees described it as an intrinsic level of confidence, and attributed it to the great pride in what past generations accomplished in the California State Water Project, noting that their designs have not been questioned since.

This lack of questioning of earlier work is clearly documented in the report on spillway drain observations during first spillway use (see Appendix F1), and more recently, in regard to the geological interpretation of foundation conditions (see Appendix C).

## 3.0 DEVELOPMENT OF DAM SAFETY IN DWR

The IFT was informed that explicit recognition of dam safety in DWR began with two senior engineers from the DSOD that were moved into O&M (Operations and Maintenance). From a 2004 memorandum [K1-2]:

> "In October 2001, the State Water Project Dam Safety Program was established for O&M to have the responsibility for managing and administering all work related to SWP dams that are operated and maintained by O&M …"

The memorandum outlines the original allocated budget for the SWP Dam Safety Program, covering 26 dams, as $1.092 million for 2002-2003 and $1.763 million for 2003-2004. There was no mandate to develop a comprehensive dam safety program; rather, this was a financial program that was separately set up to:

- "Manage and administer work projects related to SWP dams …
- Review, compile and analyze surveillance instrumentation data …
- Prepare annual structural performance reports …
- Assist field division personnel in developing, maintaining, augmenting and improving instrumentation systems …
- Conduct briefings with regulatory agencies and field divisions …
- Conduct annual and biannual dam inspections …
- Attend courses to keep abreast of technological advances in instrumentation, techniques of dam surveillance and maintenance of computer files …"

This beginning grew into a separate Dam Safety Branch, which was formed in 2008. The IFT was told in interviews that the Dam Safety Branch was formed in recognition of a critical need for additional resources to meet DWR's regulatory compliance requirements. The Dam Safety Branch Chief was an environmental engineer with a background in instrumentation. This person had been involved with dam surveillance since about 2001, and was keen to develop a dam safety program with a dedicated team. From interviews, the IFT learned that there was no dam safety training at the time, and the Dam Safety Branch has essentially self-developed since this start in 2008.

Following the 2005 failure of Taum Sauk Dam, FERC recognized the necessity for dam owners to have a specified dam safety program as a means of maintaining safe dams and preventing dam failures. FERC followed up by adding a regulatory requirement for all owners to develop formal Owners Dam Safety Programs (ODSPs) specific to their organization and portfolio of dams. DWR complied by issuing their first version of an ODSP document in March 2013. One of the new requirements was to have a designated Chief Dam Safety Engineer (CDSE), and DWR complied by placing the Dam Safety Branch Chief into this role and developing a set of required CDSE qualifications.

## 4.0 CURRENT DAM SAFETY BRANCH IN DWR

As well as handling regulatory requirements for monitoring, inspections, and reviews, the Dam Safety Branch now also initiates investigations deemed necessary by either external reviewers or the Chief Dam Safety Engineer. Investigations funded over the past years by Dam Safety Program have included:

Civil structure stability analyses (predominantly intake tower, outlet structures)

Radial gate analyses for spillways

Embankment stability analyses (including Oroville)

Flood hydrology and inundation maps

Updating seismic ground motion estimates

The Dam Safety Branch also drives capitalized projects funded by the DWR O&M Field Divisions. Examples include:

Perris Dam Remediation

Oroville Dam's River Valve Outlet Structure

Castaic Tower Remediation/Replacement

Clifton Court Radial Gate Refurbishment

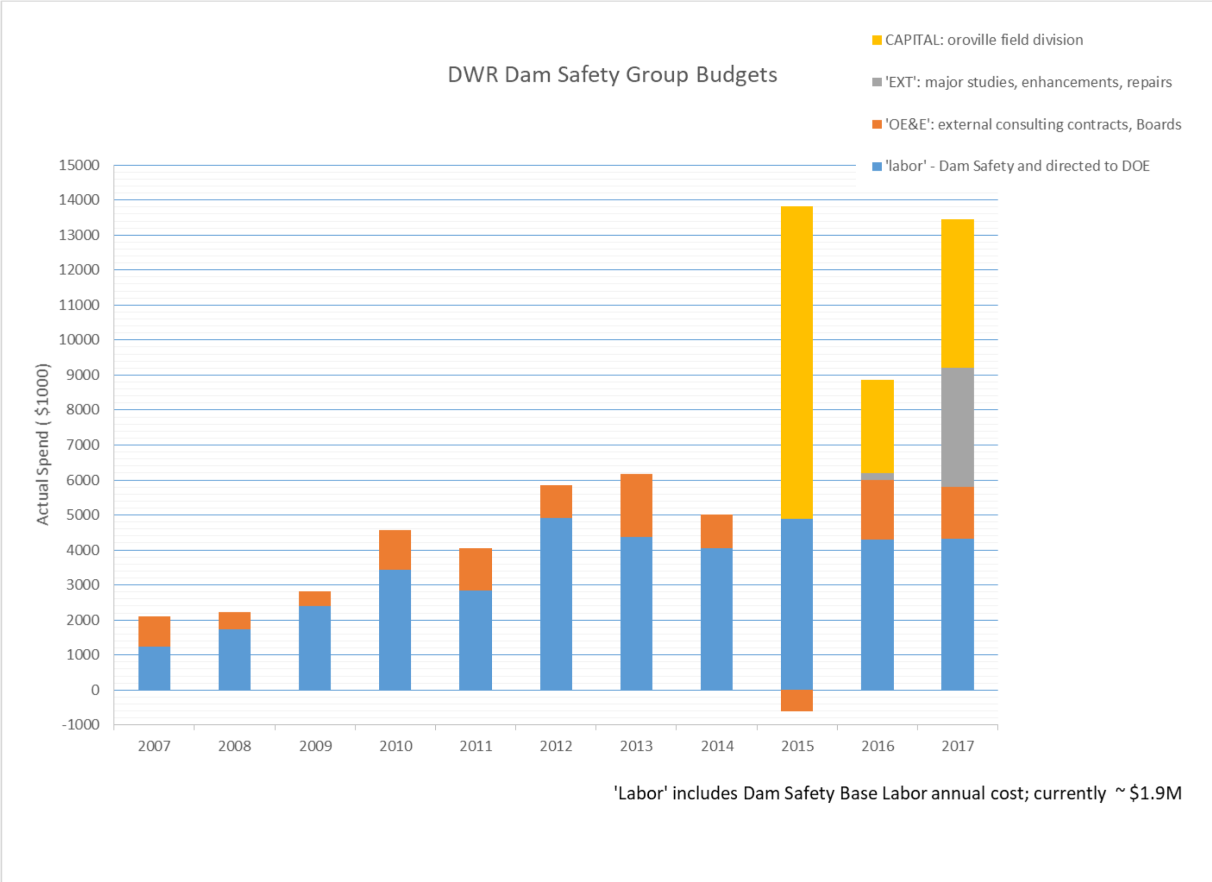
BF Sisk Dam Remediation

The IFT heard various and wide ranging budget figures purportedly available for dam safety in a number of interviews at different levels in the organization. Based on information provided by DWR in a series of spreadsheets, the IFT developed Figure K1-1 showing actual spending on dam safety, and on dam safety driven capitalized projects funded by the Oroville Field Division since the formation of the Dam Safety Branch. Funds are identified in four categories:

1. Labor: covering the burdened salaries of those in Dam Safety Branch (currently about $1.9 million), as well as the salaries of those in the Division of Engineering and elsewhere in DWR, working on dam safety initiated work, for a current subtototal of about $2.6 million. Labor also includes contractor costs, to arrive at the total as shown in the Figure K1-1.
2. Capital: covering major project work in the Oroville Field Division.
3. "OE&E": Operating, Engineering and Expense costs, covering the costs of Review Boards, external consultants, water fees etc.
4. "EXT": Extraordinary budget set up to fund major studies, enhancements and repairs.

This figure shows:

- During the first four years of the Dam Safety Branch, it was in a development phase, with little spending prior to 2012 beyond base salaries to cover surveillance, monitoring and regulated reviews.
- Average OE&E costs covering Review Boards, external consultants, water fees etc. on the order of $950,000 annually.
- Average of about $4.5 million annually in labor costs from 2012 to present.
- Little to no spending on major capital upgrade projects by the Oroville Field Division from 2007 through 2014, however this may be somewhat misleading, as capital spending for civil works was apparently allocated to the Hyatt powerplant in this period.
- The establishment of the "Extraordinary" budget in 2016, but with no significant expenditure until this past year.




Note: OE&E funding in 2015 shown as per accounting records. Actual funding from 2014 through 2016 should be averaged.

**Figure K1-1: DWR Dam Safety Group Budgets**

Of main interest is the available budget for engineering studies to address outstanding questions that the Chief Dam Safety Engineer would deem necessary to pursue. Accounting for a baseline cost of Dam Safety Branch personnel in the order of $1.9 million annually, the labor spending shown above reflects about $2.6 million annually for studies initiated under the Dam Safety Program from 2012 to present.

The funding available for investigations was augmented in 2016 by Extraordinary Funding, put in place in recognition that the previous level of funding was not sufficient. The first year of significant spending was 2017. Although it could appear that this funding was related to the February 2017 incident itself, this is not the case: the IFT was informed that there was much work planned prior to the incident, and DWR was "getting the program on track" to completing some significant analyses and evaluations. There were over 100 planned expenditures slated for 2017, including seismic analyses and stability modeling, inundation mapping, documentation updates and various inspections. Minor works such as piezometer installations and road and apron repairs were also included in this funding.

## 5.0 CURRENT STATE WATER PROJECT OWNERS DAM SAFETY PROGRAM AND ORGANIZATION

The most recent ODSP document is from 2015 [K1-3], however is out of date due to organizational changes; a 2017 update was also made available in draft to the IFT. The ODSP documents describe "the roles and responsibilities for operation, maintenance, monitoring, and regulatory oversight for 22 Department of Water Resources (DWR) dams." As per the DWR ODSP:

> "The objectives of the Owner's Dam Safety Program are to:
>
> - Maintain dams in a safe condition;
> - Provide reliable and useful instrumentation data with competent analysis, interpretation, and if needed, corrective action;
> - Train staff in dam safety to their appropriate level of responsibility;
> - Provide consistent, clear and concise documentation regarding the dam safety program;
> - Maintain regulatory compliance; and
> - Develop new and innovative ways to conduct business"

The document gives an organization chart showing the relationship and reporting requirements between various groups, and describes these as such:

> "The Division of O&M functions in the role of 'dam owner'."
>
> "In addition, O&M Division staff involved with Dam Safety include:
>
> a. Operations Branch – operate the SWP … Provides daily, visual surveillance of all SWP facilities …
> b. Engineering Branch – collect instrumentation (surveillance) data for the SWP facilities including the dams. Provide engineering support to on-site operations or maintenance staff
> c. Plant Maintenance Branch – includes mechanics and electricians that service and maintain …
> d. Civil Maintenance Branch – provide civil maintenance such as vegetation removal/control, grading, paving, concrete work for SWP facilities, and
> e. License Coordination Branch – unique to the Oroville Field Division. Provides coordination for Oroville Field Division relicensing implementation and compliance …"

The documents list specific responsibilities for the Dam Safety Branch, and provide general comments on the responsibilities of the other mentioned parties. They also provide sections covering Training, Communication and Reporting, Record Keeping, Succession Planning and Assessment/Audit requirements.

## 6.0 IFT REVIEW COMMENTS ON THE ODSP

The IFT provides the following sections in review of the DWR ODSP, based mainly on numerous interviews. These sections should not be construed as an in-depth organizational review, but rather, as comments that relate specifically to the issues being investigated as given at the start of this Appendix. These comments use the International Congress on Large Dams (ICOLD) Bulletin B154 [K1-4] as a reference point for comparison.

### 6.1 Comparative Background: ICOLD Bulletin B154

This ICOLD document presents a framework for managing the safety of dams, and

- Outlines overarching principles for dam safety management;
- Outlines the elements of a dam safety management system; and
- Provides practical guidance and enabling strategies for effective management of dam safety, and addresses on-site activities.

The ICOLD Bulletin's overall approach can be exemplified by Figure K1-2, which illustrates that any hydroelectric corporation has the potentially conflicting dual responsibilities of safety and production. The corporation needs to find the appropriate balance between safety and production, and the upper management needs to be assured that this has been undertaken.

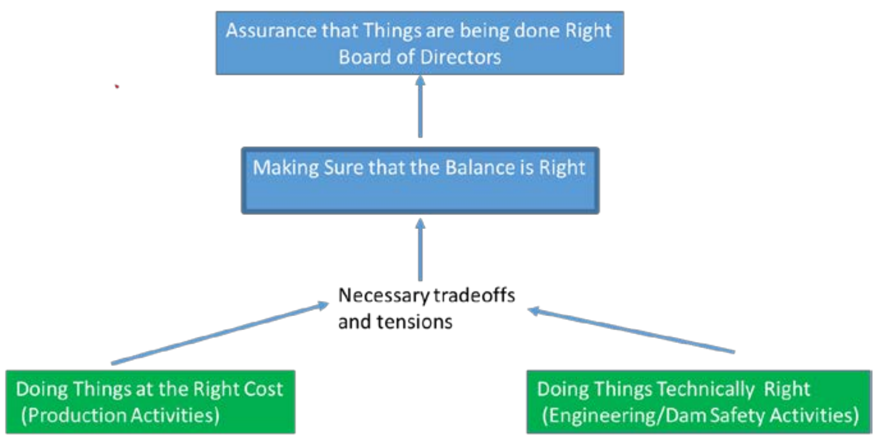



**Figure K1-2: General Dam Safety Roles in a Corporation**

There are many different organizational structures that can support the functions necessary for an effective dam safety management system, as long as these overarching roles are filled. However, ICOLD B154 notes that the responsibility for dam safety is "normally delegated to an appropriately qualified (in the engineering and scientific sense) senior executive or manager".

The Bulletin suggests that the executive recognize the role of Dam Safety in the organization by way of a statement or policy. The executive needs to approve this statement or policy, which should, among other items, clearly indicate that any immediate dam safety requirements have priority over any other aspect of the business.

Some additional points for comparison against DWR include (paraphrased from ICOLD B154):

Implementation of dam safety functions broadly includes:

- Safety assessments
- Asset Portfolio Management
- Risk reduction measures and safety improvements

All of the organization's goals, strategies, plans and objectives should be considered collectively in a coherent manner. The formalization of the process is achieved by development of a series of policies, procedures, directives and instructions, with logical and functional links between the individual documents.

If the management system is not integrated, any areas of overlapping responsibilities, omitted or poorly defined responsibilities or conflicting objectives must be carefully examined. At the lower organizational levels, the system should specify how different activities between the groups that are involved in a single process are planned and managed effectively.

There must be a clear allocation of tasks, responsibilities and authorities. Sufficient authority must be given to all personnel involved with dam safety to identify and report any safety issues, initiate actions to prevent incidents and accidents, and to initiate, recommend or develop solutions. Those who are responsible for oversight must have accurate and timely information about all issues that could affect dam safety. The system should define the competency requirements for staff at all levels.

The ultimate responsibility for the management system rests with senior management.

The management system should establish a monitoring, measuring, analysis and evaluation system for the assessment of dam safety performance and the effectiveness of the system itself. This should cover all elements of the dam system, to provide early warning of inadequate performance.

The directors of the dam owner typically appoints a specific individual to be responsible for directing and overseeing the management of all activities necessary for dam safety. They should be able to see the tensions between "dam safety" and "production" activities to satisfy themselves that the trade-offs between these somewhat competing objectives are being dealt with properly.

A registry of all assets linked to dam safety, including maintenance plans, should be developed.

Internal assessments should be carried out by senior management, who should also establish a schedule of independent audits.

Continuous improvement should be one of the objectives.

## 6.2 General Governance and Policy Considerations

The DWR ODSP document is much less detailed than would be expected from an organization dealing with major dams. The dam safety roles, responsibilities and authorities for each and every position on the organization chart should be defined and documented, as well as how the groups are expected to interact with each other in the safe retention and passage of water, who must provide the final technical position on all risk issues, what training and background they are expected to have, etc.

The DWR ODSP is silent on both ends of the spectrum in regard to dam safety: the overall responsibility and authority for dam safety in the organization on one end, and the specifics of the work items pertinent to dam safety on the other. The ODSP does not specify who has overall accountabilities to ensure the required tasks are delegated and undertaken. The IFT is aware of detailed and separate Dam Safety Governance Manuals, and Implementation Manuals, in use at other utilities. Such documents are not regulatory requirements, but considered necessary by these organizations to fulfill their dam safety due diligence.

Although not identified as underlying principles or policies, some overall statements are made in the ODSP regarding DWR's approach to dam safety [K1-3]:

> "State Water Project Dam Safety is a shared responsibility that involves offices and personnel throughout DWR.
>
> The expectation of the program includes maintaining safe operation as the first priority. Hydroelectric power generation, water deliveries, or other business objectives shall not compromise dam safety nor regulatory compliance. Employees shall be empowered to report dam safety concerns without fear of reprisal."

The IFT was surprised by the range of opinions given in various interviews as to where overall responsibility for the safety of water retaining structures lay, ranging from Director to Deputy Director to Division Chief. One DWR executive opined that this responsibility rested with the regulator (DSOD) and not the dam "owner."

### 6.3 Investigations

Of particular concern, the ODSP does not lay out who is responsible for identifying and prioritizing new risks as they emerge, and doing the same for potential deficiencies, and then following through with appropriate investigations. The ODSP does not outline any specific means to properly follow through with particular questions that may arise, such as why the spillway drains flow when the chute is in operation, and whether this should be considered "normal." The IFT is aware that these questions were asked, but investigations were not followed through to any satisfactory conclusions. The same situation occurred in regard to the erodibility of the bedrock. The implications of the herringbone cracks were also never investigated, although repeatedly noted in DWR, DSOD, and FERC inspections.

The IFT was told that there are investigations proposed by the Dam Safety Branch that have not made the DWR Director's Board or FERC recommended lists. This is due to a general lack of resources, with the result that "things that aren't broken" are not being investigated. There is also no tracking or prioritizing of issues raised by others in the organization such that they can be addressed at a time when resources are available (for example, during a Potetential Failure Modes Analysis). This would allow issues that were repeatedly noted (such as described in the previous paragraph) to be formally addressed. Once addressed, the conclusions could be reviewed in subsequent studies to determine if they are still valid based on current knowledge of potential failure modes and dam safety issues.

General work overload and pressures to just keep up with regulatory requirements were given as main factors in not pursuing these proposed investigations further. However, the IFT notes that investigations, whether undertaken on a Board recommendation, or deferred due to lack of resources, are essentially ad-hoc, and are not being vetted through a system of documentation and risk prioritization, to ensure that the most important issues are receiving timely attention.

### 6.4 Organization

The IFT was supplied with literally tens of organizational charts for each Division, Branch, and Section of DWR over the years, as well as a series of Water Resources Engineering Memoranda (WREMs) that outline in general terms the relationships between these various groups during design, construction and operational phases of the DWR assets. However, the IFT first attempted to build its own chart on the basis of interviews with DWR staff, to assess the level of understanding "from the bottom up." It was only with significant effort and repeated conversations with various DWR staff that the IFT was able to produce an organizational chart that (we believe) accurately depicts all Divisions and groups that should be involved in SWP dam safety. Refer to Figure K1-3 at the end of this Appendix. This figure also shows regulatory structure, which is discussed in Appendix K2. Only after this chart was initially drafted did the IFT allow itself to review the organizational chart given in the ODSP.

Comparisons of the ODSP versus draft IFT charts were revealing. It was apparent that various levels of supervision were changing on almost a year-to-year basis, and staff were not fully aware of, and somewhat confused by, the current organizational structure. The IFT noted missing administrative levels when comparing the charts, and also additional levels, due to organizational changes since the ODSP was published in mid-2015. There were also groups on the ODSP chart that were not mentioned in any first discussions with the Dam Safety Branch. The IFT was also told that the Dam Safety Branch did not deal directly "a whole lot" with Operational groups involved with water management.

The IFT found it unsettling that the Dam Safety Branch would not have a direct relationship defined with other groups shown on the ODSP chart. In general, most of those DWR persons interviewed had difficulty explaining and understanding the organizational "who's-who" with respect to dam safety responsibilities and accountabilities. Figures K1-4 and K1-5 (at end of this Appendix) compare the final IFT chart showing what we consider to be all groups associated with the overall system of dam safety within DWR against those positions identified in the current (2015) and draft ODSP documents.

### 6.5 "Ownership" of the Assets and Inter-Group Relations

In the current DWR organization, there is a separation of most of the civil/geotechnical engineering resources and electrical/mechanical engineering resources into different Divisions. Civil/geotechnical engineering mainly resides in the Division of Engineering (DOE) which is considered to be a "service provider." Electrical/mechanical engineering resides under the Division of Operations and Maintenance (O&M) and functions in the role of "dam owner" as per the ODSP. This arrangement was put into place following a 2013 study [K1-5] that advocated for the integration of all mechanical, electrical, and inspection services. Subsequently, DOE's Mechanical and Electrical Engineering Branch was moved to the O&M Division in July 2013, followed by DOE's Equipment and Materials Section of the Construction Branch in July 2014. Thus, all electrical/mechanical personnel and services are now within the O&M Division. This reorganization was to utilize the electrical/mechanical resources more effectively and efficiently. There is now a Service Level Agreement [K1-6] that defines the joint processes, working relationships and performance expectations between O&M and DOE. The document includes a chart that identifies the lead organization for services within each phase of a project and a checklist for detailed tasks.

The IFT found through numerous interviews that this division of engineering resources has led to misunderstandings and resentment between the Divisions. From various interviews and written comments by DOE staff, the IFT noted the following repeated themes:

- O&M's role as "owners" of all SWP assets, as well as the "operators," gives the impression that O&M is the sole decision-maker for what scope becomes a project, how much budget is allowed, and what priority is assigned to each O&M project. Thus, Civil Engineering projects (such as spillways and canals) are seen as having had less priority than Mechanical and Engineering projects (such as pumping and power plants).
- The O&M engineers and managers are seen to control the budget and schedule, without having a full understanding of the technical work or dam safety concerns. One interviewee described it as both a frustrating and humiliating "master-slave relationship," where the attitude of many O&M managers is to either accept their decisions or accept their giving the work to outside consultants.
- As O&M Field Divisions take ownership of their geographic areas, they are seen as being run "like little kingdoms," and as such, the Field Divisions take on their own designs and contracting for smaller work.
- O&M has very few civil engineers, and those on staff perform little or no design and construction work. Thus, O&M culture is seen as being absent of civil engineering input and respect.

In general, most interviewees from DOE believed that the disparity between electrical/mechanical "owners" versus civil/geotechnical "service providers" has significantly influenced prioritization of work, and that work up until now has been prioritized only within the various Divisions, not across the entire organization. It is recognized, however, that there is a longer term plan for the Asset Management Program Office to take the lead with prioritization of maintenance and capital projects.

Difficulties with the relationship were not solely cited by those in DOE. DSOD personnel related their belief that the O&M/DOE relationship is "disconnected," and they are unclear as to how deficiencies are prioritized. As a counterpoint to the above, they noted that DOE often seemed to have overall control and authority for a project, and mused whether or not O&M had enough influence on technical decisions in their dual role as the "owner" and "operator."

In general, the IFT gained the impression through many interviews that, certainly in the past, there appears to have been limited sharing of information between groups, a reflection of the relatively strict hierarchy and group structure, with everyone in their separate box. Numerous interviewees were of the opinion that this was changing, and believed that the relationship between Operations and Dam Safety has become better over the years as training and trust has evolved. The IFT learned for example that Dam Safety (O&M) had organized regular meetings between O&M and DOE geologists, and lunches once per month. However, many interviewees also noted that the strain between O&M and DOE stretches back over decades, and were less optimistic that the situation is improving.

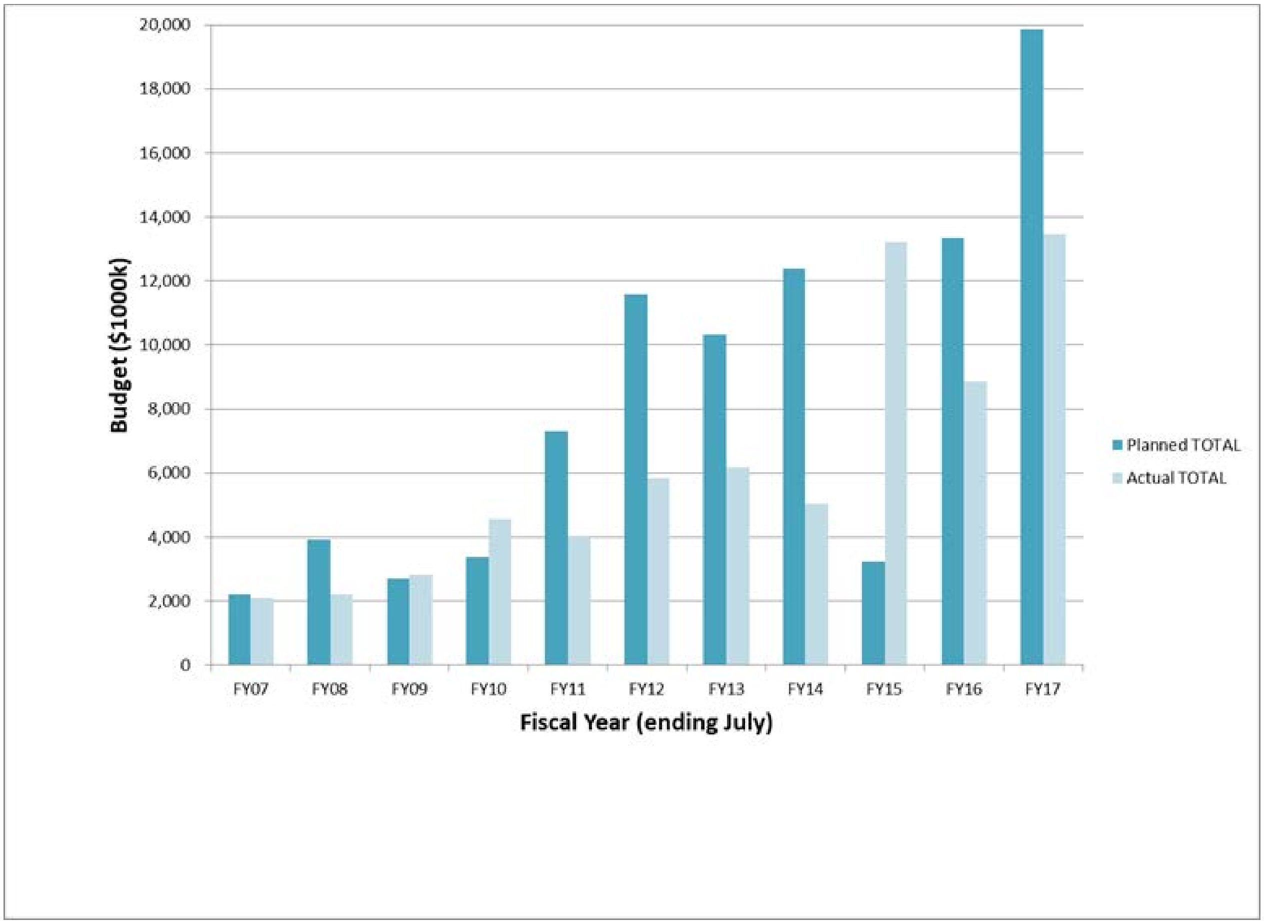
DWR personnel themselves recognize that at least part of the ongoing issues relate to the overall lack of dam safety governance as previously discussed. Interviewees recognized the “grey area” between Operations and the DOE, and the necessity for clarity in regard to the roles and responsibilities between the individual positions.

### 6.6 Resources

The IFT was told in a number of interviews (at different administrative levels) that dam safety investigations and capital upgrades were not being hampered by available budgets, but rather, by available human resources. In effect, there has always been more budget than can be spent. This claim was examined by the IFT. Figure K1-6, based on data provided by DWR, exemplifies that the Dam Safety Branch has been historically resource constrained, and actual spending is typically less than budgets.

To help with this resource constraint, the IFT was informed that the Dam Safety Branch was expanded from 6 to 9 staff engineers in 2016. However, there is still a major resource constraint both within the Dam Safety Branch and elsewhere. The Dam Safety Branch had developed and shared lists of investigations that they want to pursue, but that had not made the FERC Part 12D recommendation list due to the resource constraints. The IFT was told that there were times that resources were pressured to respond to regulatory requests that were felt to be of lower priority than other work that subsequently had to be deferred.

The IFT also heard from several sources that although the Senior Enginers in the Dam Safety Branch are “the most hard-working in DWR,” they just barely keep up with regulatory demands. Seniors are said to be working 50-60 hours per week, but are not compensated for overtime.




Note: 2015 shown as per accounting records. Actual totals from 2014 through 2016 should be averaged.

**Figure K1-6: Planned vs Actual Spending**

It is apparent from the IFT interviews that the Dam Safety Branch does not necessarily have the background expertise to be knowledgeable on all aspects of engineering involved with dam safety. The IFT was told that most of the studies, or contracting for dam safety studies, are being done in DOE, and assigned to staff based on availability. There is a reliance on the DOE to have the necessary expertise, and to remain abreast of engineering progress and developments that could affect dam safety evaluations. Such areas cover a wide range of topics, from hydrology and seismicity to detailed design methodologies. While interviewees indicated that DOE staff have the option of taking training, there has been no apparent consistency in assigning dam safety studies to specific engineers, and therefore, no particular incentive for DOE staff to develop an understanding or knowledge of state-of-the-art dam safety practices. In the absence of assigning dam safety work to dedicated staff in DOE, the IFT believes that there have been many missed opportunities to identify issues related to the Oroville spillway that resulted in the February 7, 2017 issues. For example, as noted elsewhere, engineers assigned to make repairs to Oroville spillway since its construction, failed to recognize that drain flows and cracking that seemed “normal” were actually signs of design deficiencies in need of correction to prevent a failure.

## 6.7 Role of Chief Dam Safety Engineer

The ODSP lays out the qualifications for the CDSE. From a technical perspective, these qualifications are simply:

> "The position requires the ability to function as a competent specialist in dam safety;
>
> Valid registration as a civil engineer; and
>
> Minimum of two years of experience performing the duties of a Senior Engineer (following a minimum of four years in more junior engineering roles)."

The current CDSE, although quite competent in his role as head of surveillance and monitoring, fully dedicated to his role as CDSE, and having gained much experience and insight since being posted to the position, did not have significant experience relevant to dam design, construction or safety prior to his current role. In contrast, the IFT is aware of a current talent search for the equivalent role in another major utility that is requiring a minimum 20 years engineering experience, which must include design and construction for hydroelectric projects, as well as monitoring and surveillance.

As per ICOLD B154, the specific individual responsible for directing and overseeing the management of all activities necessary for dam safety must be in a position to see the tensions between "dam safety" and "production" activities. They must satisfy themselves that the trade-offs between these somewhat competing objectives are being dealt with properly. It is clear that the position of the CDSE on the DWR organization chart, currently down four layers of administration from the Deputy Director of the SWP, is not in such a position.

## 6.8 Communication with Upper Management

Communication of dam safety issues is of some concern due to the relatively rigid chain of command within DWR. There is an available direct line of communications available from the Chief Dam Safety Engineer to the Deputy Director for the SWP, however this has never been thought necessary to utilize. The IFT was told that the Deputy Director does not receive regular updates on dam safety risks. An annual report covering data reviews and inspection recommendations is submitted to the Division Chief, but the authors of the report were unaware of whether this was forwarded to the Deputy Director. In the opinion of the IFT, at the very least, the Chief Dam Safety Engineer should be making quarterly presentations to the SWP Deputy Director where the report and matters of dam safety could be discussed. This should be paired with periodic updates related to dam safety issues and program progress. Without such communication, DWR appears to be relegating dam safety to a relatively minor role in the organization. During interviews with FERC personnel, the observation was made that the CDSE position seems to be bypassed during many decisions.

Although having the potential for direct communications in an emergency situation is commendable, in conversations with FERC personnel, it was clear that one goal of requiring an Owners Dam Safety Program was to establish regular and free communication between the CDSE and upper management, which holds the overall responsibility for dam safety in the organization.

This is currently not the case in DWR Interviewees from DSOD opined that DWR needs to build up their expertise and base strength in-house, in order to have “a loud voice” when speaking to management.

FERC personnel noted that an external management audit, had one been held, would have commented that although communication within DWR has improved, there is still a lack of a regular line of communication between the CDSE and upper management. The IFT notes a significant difference of opinion was expressed by DWR executives, who did not see any issues with the ease of communication regarding dam safety issues.

### 6.9 Training and Continuous Improvement

The ODSP stipulates that engineers in the Dam Safety Branch will complete training which includes:

- United States Bureau of Reclamation course on Safety Evaluation of Existing Dams (SEED)
- Association of State Dam Safety Officials (ASDSO) and American Society of Civil Engineers (ASCE)
  - Annual conferences
  - Webinars
  - Specific training seminars on slope stability analyses, conduit design, instrumentation and inspection of dams
- FERC training and workshops for
  - Risk-informed decision making
  - Part 12D Safety Inspections etc.

There is no such stipulation for technologists or others in dam safety engineering and surveillance roles.

Since 2008 there has been a sincere effort by DWR to provide training to staff; for example, now at least 2/3 to 3/4 of DSB personnel have gone through the USBR Safety Evaluation of Existing Dams course. However, there have been a number of limitations, particularly in getting approval for out-of-state travel to courses or seminars. The IFT was informed that requests for out-of-state travel, including specific dates and locations, need to be submitted a minimum of a year in advance. This is of particular concern: no dam safety organization can be expected to grow and mature without exposure to national and international peer experience and continuing conversation. Many dam safety conferences and meetings are not organized a year in advance, such as those by the Dam Safety Interest Group of CEATI, an international organization of about 80 dam owners and regulators who share ideas in general meetings and workshops twice a year in various locales across the United States. DWR is a member of this organization, however its participation has been hampered in the past by the out-of-state travel rules. It was noted in one executive interview that for DWR to be world class, it needs to be connected to the outside world, but that this was extremely difficult within the rules of a State agency. The IFT notes that one option would be to place more emphasis on bringing global external experts into the DWR organization to provide information and training sessions that are open to staff in both the DSB and DOE.

### 6.10 Auditing and Benchmarking

The DWR ODSP lays out a requirement for an external audit of the dam safety management processes every 5 years. The first required audit (since the start of the ODSP in 2013) was being organized in early 2017 just prior to the Oroville spillway incidents, and was deferred. The 5-year dam safety reviews required by DSOD/FERC do not cover management, organizational or budgetary considerations, as they focus on the safety of the dams themselves. Thus, there has never been any documented audit of any kind performed on the DWR dam safety management system. The IFT considers this to be highly unusual for such a large utility. Irrespective of regulatory requirements, management audits are held by many large utilities on a regular basis.

FERC, recognizing the lessons to be learned from the Taum Sauk failure, developed an evaluation form designed to assist dam safety owners in judging the effectiveness of their programs. The IFT was informed that this was informally referenced by the Dam Safety Branch, however there was no documentation. It is clear that in DWR, any requirements for audits are driven by regulatory requirements only, not by due diligence considerations.

## 7.0 OVERALL ASSESSMENT OF THE DWR DAM SAFETY MANAGEMENT SYSTEM MATURITY

To provide an assessment of the DWR program in terms of the elements being explored in this Appendix, the IFT compared dam safety management and governance at DWR to the principles as laid out above in ICOLD B154 and by way of a relatively new tool in dam safety, "maturity matrices."

The owners/regulators group CEATI has developed a series of 10 Maturity Matrices for self-evaluation of key dam safety program elements, that define five levels of maturity for each element, ranging from "Needing Development" to "Leading Edge." Any program would be expected to have a spectrum of results over the matrices, perhaps encompassing all five levels for various different elements.

As described in CEATI literature [K1-7]:

> "The following dam safety program elements are used:
>
> - Surveillance
> - Flow Control Equipment (FCE)
> - Reservoir Operations and Public Safety Emergency Preparedness
> - Dam and Water Conveyance Structure Maintenance
> - Managing Dam Safety Issues (DSIs)
> - Audits and Review (Program and Dam/FCE)

- Training and Education
- Information Management
- Governance

The Maturity Matrices are a powerful tool to:

- Evaluate the effectiveness of an Owner's dam safety program.
- Identify program strengths and areas for improvement.
- Communicate program strengths and areas for improvement to managers and other key stakeholders.

> The Maturity Matrices can be used to evaluate dam safety programs, initially, to identify areas for improvement, and periodically, to track subsequent improvements. The primary benefit from using the Maturity Matrices, is expected to be the improved understanding of a dam safety program, across the whole range of activities that influence its effectiveness."

The IFT, in the investigation as to whether the general practice and culture of dam safety within DWR could have had influence in the timelines to failure, referred only to the 10$^{th}$ CEATI matrix, which addresses "Governance." This is provided as Figure K1-7 (at end of this Appendix), with highlighted elements that the IFT consider to be representative of DWR. The Governance elements were mainly "Needing Development" to "Intermediate", with regulatory compliance elements rated at "Good Industry Practice." There were no elements that were considered to be "Best Industry Practice" or "Leading Edge."

In general, based on this matrix, and the ICOLD Bulletin as previously noted, the IFT considers the DWR dam safety program to be somewhat immature when compared to others worldwide. "Dam Safety" as a concept has changed over the past few decades. Originally thought of as principally monitoring and surveillance specific to dams, it continues to develop into an overall safety philosophy encompassing the full management and functional safety of reservoirs.

Consider the 2004 definition of Dam Safety given by the Federal Emergency Management Agency [K1-8]:

> "*Dam safety* is the art and science of ensuring the integrity and viability of dams such that they do not present unacceptable risks to the public, property, and the environment. It requires the collective application of engineering principles and experience, and a philosophy of risk management that recognizes that a *dam* is a structure whose safe function is not explicitly determined by its original design and construction. It also includes all actions taken to identify or predict deficiencies and consequences related to failure, and to document, publicize, and reduce, eliminate, or remediate to the extent reasonably possible, any unacceptable risks.
>
> *Dam* : An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material, for the purpose of storage or control of water." [italics added for emphasis]

This rather limiting definition of dam safety from 2004 has not caught up to leading practice. It can be compared with words from the 2011 ICOLD Bulletin 154 [K1-4] which specifically includes controlling all flows through and around the dam, and includes consideration of supporting safety critical systems:

> "The fundamental dam safety objective is to protect people, property and the environment from harmful effects of misoperation or failure of dams and reservoirs: This objective is achieved by retaining the stored volume of water and controlling all flows through and around the dam within specified limits determined through the approvals and licensing process established by government. "Misoperation" involves any departure from the design norms for safe operation of any part of the dam or its safety critical systems."

In many ways, the DWR Dam Safety Program currently remains focused on simply surveillance and monitoring of dams. This opinion was echoed in interviews with regulatory personnel. The other critical dam safety functions (as listed by ICOLD) of Asset Portfolio Management and Risk reduction measures and safety improvements are still under development. This was evident in numerous interviews conducted by the IFT, and is exemplified by the current organization chart and ODSP:

- There is no one person clearly identified as the executive that takes accountability for Dam Safety. In the Oroville spillway incident, it is the DWR Director that took charge, however the Director is not apprised of dam safety matters on a regular basis. "I have yet to see a Director at a 5-year external regulatory reviews of dam safety … there are Deputy Directors there … they let Division Chiefs take care of it (dam safety)."
- Dam safety information is passed from the CDSE through two layers of management to the "owner" of the ODSP (the Division Chief of O&M), and then on to the SWP Deputy Director. In many mature dam safety programs, the equivalent of a CDSE is considered the one technical authority for dam safety, reporting directly to the executive who has been delegated the dam safety overall accountability for the organization. The IFT believes that DWR should consider such an arrangement.
- The CDSE is the one person in the organization that should have an overview of dam safety risks, and as such, must have a large role in the management of the large civil engineering assets. The IFT was told that the Asset Management Program Office has just relatively recently been established and is "in its infancy" and somewhat "still aspirational." The Office has a good rapport with the Dam Safety Office – but the existence of Asset Management is not yet noted on the ODSP organizational chart. The relationship between the two groups is very important, and this must be allowed to grow in the future. The role of dam safety risk and asset manager should be at least on an equal footing with other asset managers.
- Any dam safety management system, regardless of organizational structure, must share numerous responsibilities across many groups. An ODSP must clearly lay out each of these responsibilities for every identified group on the organization chart that is specifically

produced to define dam safety management, and each group must have a clear understanding. This is not the case presently at DWR; although produced with some thought and care at the working level, the ODSP and corresponding organization chart appear to simply "check a box" for regulatory requirements.

The role of dam safety is clearly still in development within DWR and has made good progress over the past few years. The IFT believes that this can be expected to continue with the assistance of the newly formed Asset Management Program group, however there is much to be yet accomplished.

## 8.0 COMPARISON WITH OTHER UTILITIES

The IFT asked the regulators for their opinions of how DWR's dam safety program compares with other large dam owners in California and elsewhere. From interviews with FERC personnel, the IFT gained the impression that DWR was seen to be about "the middle of the pack," with good personnel and intentions, but having particular challenges in being part of a government agency, and with a large backlog of issues requiring prioritization.

Interviews with DSOD personnel generally backed up the above overall impressions. They noted a general complacency and a willingness of DWR to simply meet the regulatory requirements, rather than being as proactive and leading as some other California dam owners. However, they also noted that the majority of dam owners wait for DSOD to provide direction. This is perhaps a reflection of the smaller size of most other owners. In another general comment regarding dam owners in California, it was stated that there is rarely a manager above the Chief Dam Safety Engineer level who truly understands dam safety, including in DWR.

Judging by the state of dam safety maturity within DWR as given in the preceding section, there are apparently many owners in California requiring significant development in the practice of dam safety.

## 9.0 DWR SELF-OPINION ON STATUS OF DAM SAFETY

When asked their opinion on the DWR Dam Safety Program during interviews, there was a range of answers from DWR personnel – ranging from "world-class," "excellent," and "nothing necessary to improve it institutionally" (usually at higher administrative levels) to "middle-of-the-pack" and "some areas need improvement." This is not necessarily surprising; as was previously noted, any program would be expected to have a spectrum of results depending on various elements of the program, and each level of management will have their own viewpoint. In general, however, the IFT believes that there is much room for development of self-awareness at supervisory levels within DWR.

Many others saw the program as the IFT does; maturing rapidly and on the right path, but late to start. It was noted that all of the SWP civil infrastructure was constructed in about a 7-year timeframe and is now showing its age. Many felt that the DWR is "behind the curve" as an organization in terms of having the resources to address the growing backlog of issues. Others thought that management was on the right path to "maturing" the dam safety program and adding focus to civil infrastructure, but noted the difficulties of State service. One interviewee compared DWR to turning an aircraft carrier …"it will take time to mature the program."

## 10.0 SUMMARY

DWR is proud of its engineering heritage, and there is a general culture of not questioning past work. Unfortunately, this does not take into account that our general understanding of dam safety risk has evolved over time, and the original designers may not have been aware of the same concerns we have today. The organization is also very hierarchical, with a well-defined and somewhat strict chain of command, but without a corresponding list of dam safety responsibilities and accountabilities along this chain of command. Dam safety as a concept within DWR is clearly driven "bottom-up." Throughout its development, DWR was in no way leading dam safety practice, but were instead reacting to regulatory pressure. In contrast, USACE, USBR and some major utilities had well established dam safety programs for literally decades before DWR. There is no strong overarching "top-down" ownership of the program; it is left almost solely to the Chief Dam Safety Engineer, although this person is not in a position to best influence either investment or emergency decisions. There is no one DWR executive that has been specifically charged with the overall responsibility for dam safety. Regular communications regarding dam safety issues with upper management are lacking. The CDSE should be reporting directly to whichever executive has overall dam safety responsibility, so that this executive is fully aware of dam safety concerns in any emergency situation.

Although DWR established a dam safety regulatory body in 1929, which is now regarded as the leading state dam safety program in the United States (Appendix K2), it took DWR more than another 70 years as a dam owner to establish an internal dam safety group that the IFT considers to be still in development. This demonstrates to the IFT a general "hands-off" approach to dam safety at the Director and Deputy Director level within DWR. Although maturing rapidly and on the right path, the development of a dam safety program was very late to start. Its development has been hampered by strict State organization travel rules that restrict attendance at conferences and meetings of other dam owners. If travel restrictions cannot be relaxed, the IFT believes that more emphasis must be placed on bringing global external experts into the DWR organization and internet-based training programs to provide information and training sessions.

It is only very recently that appropriate levels of spending were being achieved; as such, the "backlog" of dam safety issues requiring investigation likely appeared almost overwhelming to those at the working level. FERC personnel affirmed that "DWR has a lot to prioritize." It will be imperative that a documented system of prioritization with the ability to track progress be established and rigorously followed, coupled with additional resources, for DWR to appropriately address its current risk portfolio.

The separation of main civil/geotechnical and electrical/mechanical engineering resources into "service providers" vs "owners and operators" has set up DWR to treat the risks associated with water retention and passage quite differently from those associated with power production. There is no one in a position of authority specifically tasked with ensuring that the "balance is right." There is a reliance on DOE to have the expertise in all aspects of engineering associated with dam safety, but no apparent motivation to provide and undertake the training necessary to attain and maintain the appropriate level of expertise.

The IFT also believes that the culture within DWR, and the relative immaturity of the dam safety program, had a significant influence on past investigations into spillway condition and performance, and on the outcome of the critical decision making during the Oroville spillway incident. This is further explored in Appendix L. The issue was not necessarily the organizational structure itself, but rather, the overall relatively immature development of dam safety within DWR. In general, a significant "top-down" proactive effort to advance dam safety culture and awareness beyond simple regulatory requirements has barely begun, and judging by interviews with regulatory personnel, this is not uncommon among dam owners.

## 11.0 REFERENCES

[K1-1] Progress of the Department of Water Resources, 1961-1966, Verbatim transcript of a meeting of the Executive Staff of the Department of Water Resources December 20, 1966, at which staff officers reported on Departmental accomplishments during the period January, 1961 to December, 1966 under Director William E. Warne.

[K1-2] State Water Project Dam Safety Program, R. Sanchez, March 15, 2004, file 20.30.178.

[K1-3] Owner's Dam Safety Program, FERC Project No. 2100 and No. 2426, Department of Water Resources, State of California, August 2015.

[K1-4] Dam Safety Management: Operation Phase of Life Cycle, International Congress on Large Dams (ICOLD) Bulletin B154, 2011.

[K1-5] Business Case: Realignment of Mechanical and Electrical Engineering Resources for State Water Project Facilities, DWR, May 2013.

[K1-6] Department of Water Resources, Service Level Agreement (SLA) Between the Division of Operations and Maintenance and Division of Engineering, Established November 17, 2014 (V2.0 September 2016).

[K1-7] Using Maturity Matrices to Evaluate Dam Safety Programs, Dam Safety Interest Group (DSIG), CEATI International Technical Brief, Project No. T132700-0234, 2015.

[K1-8] Federal Guidelines for Dam Safety, Glossary of Terms, April 2004, FEMA. Available at https://www.fema.gov/media-library/assets/documents/3904.

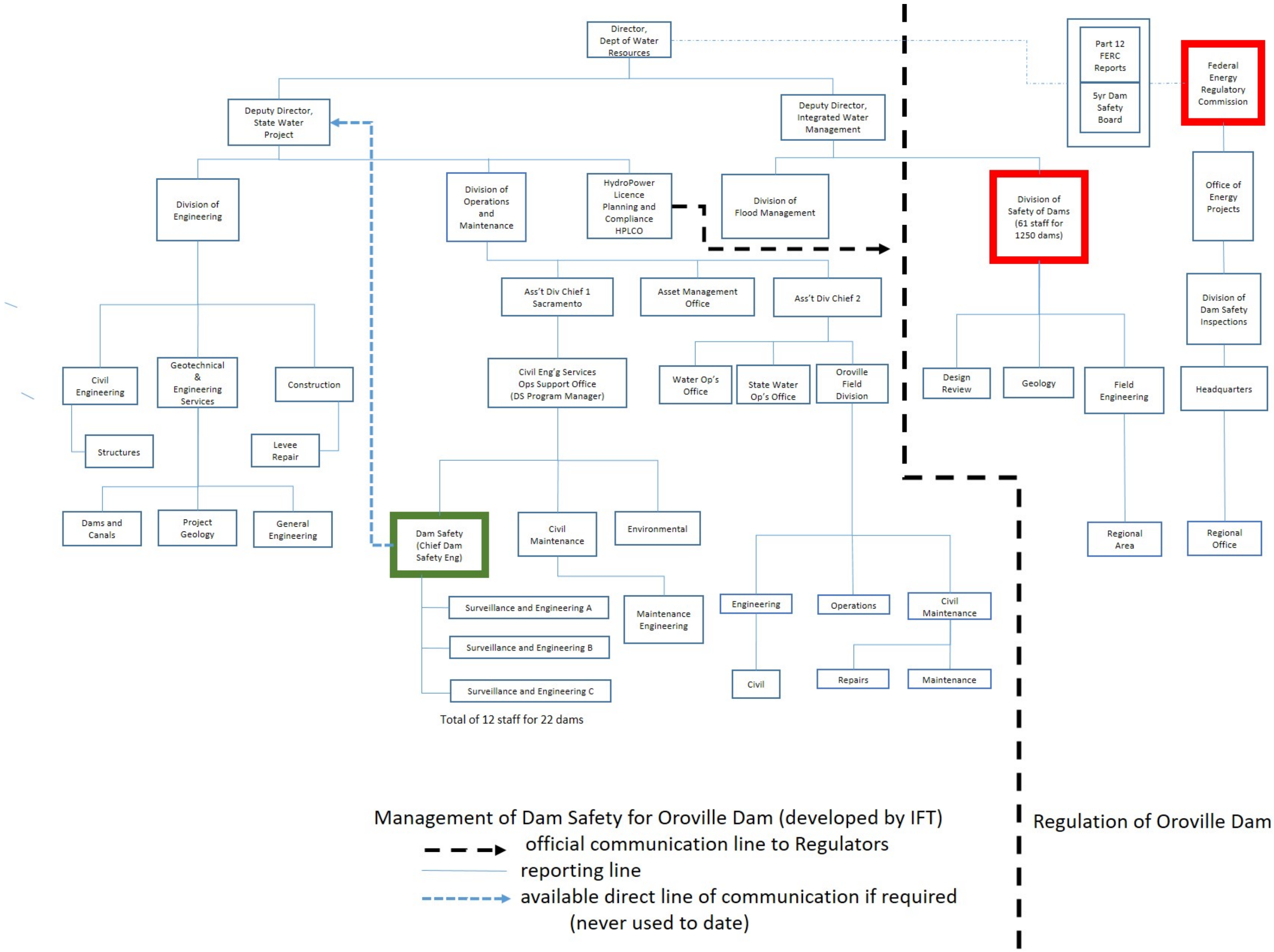



**Figure K1-3: IFT Interpretation of Dam Safety Management for Oroville Dam**

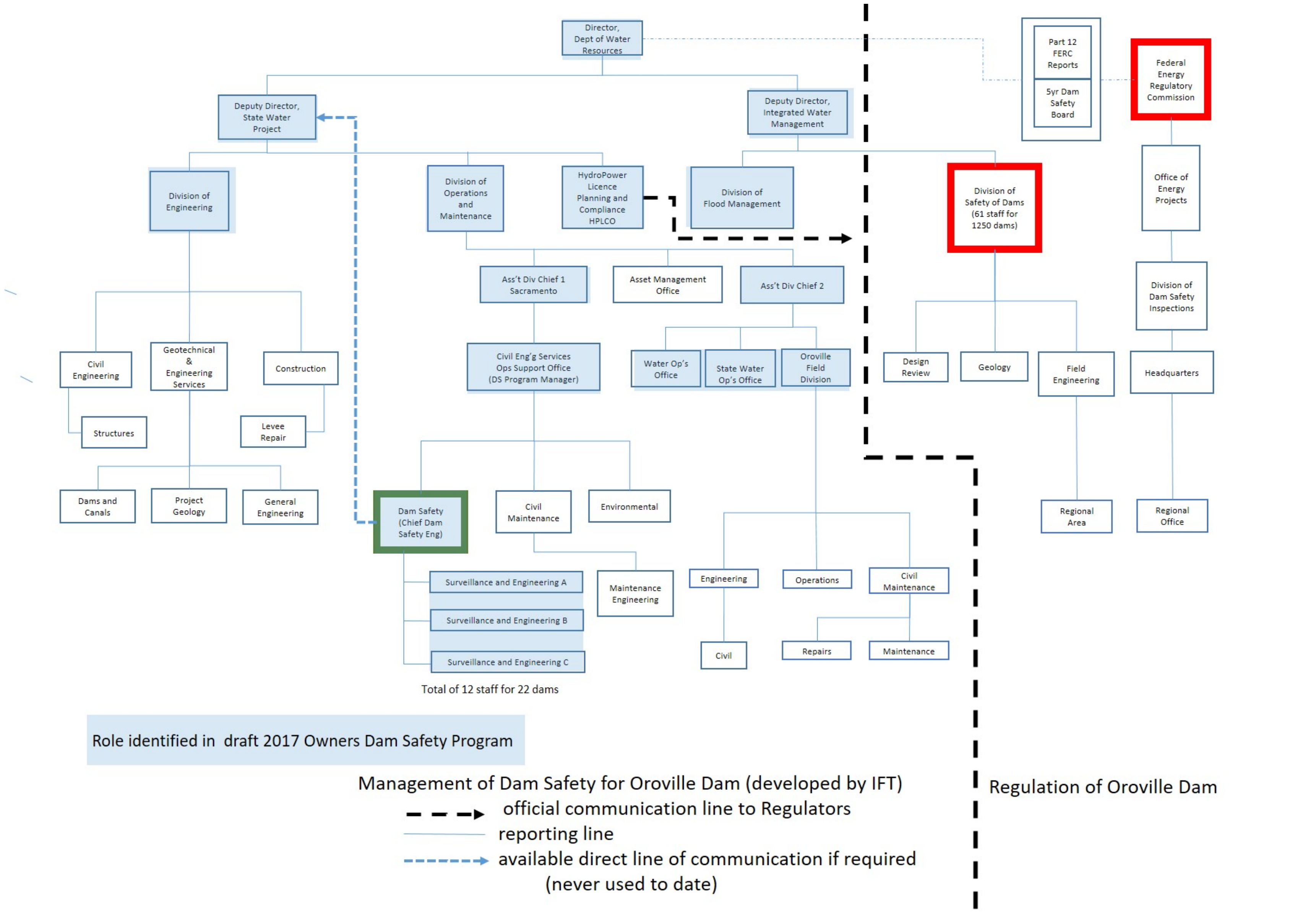



**Figure K1-4: Roles in Dam Safety Management identified by DWR in 2017**

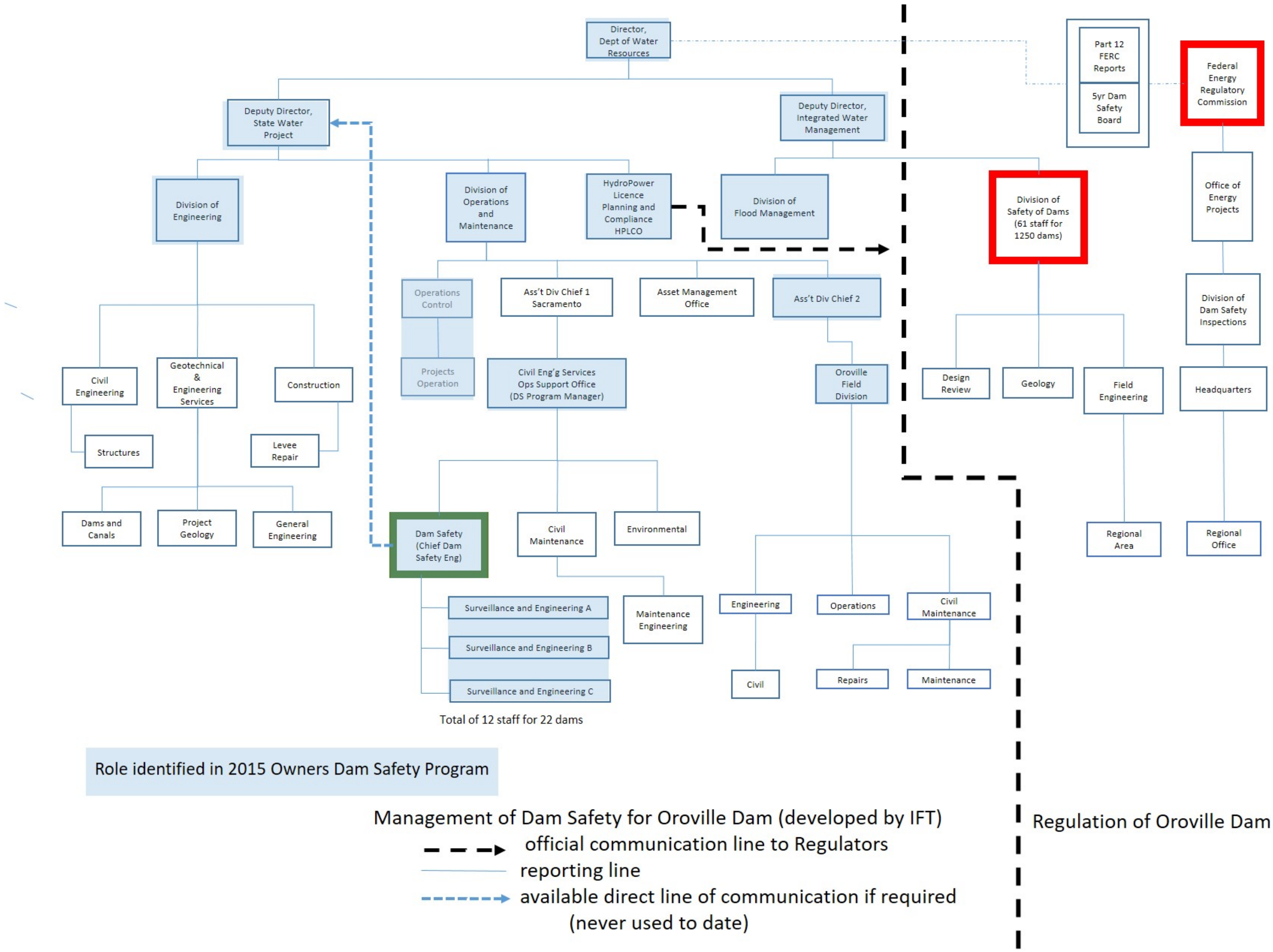



**Figure K1-5: Roles in Dam Safety Management identified by DWR in 2015**

Using Maturity Matrices to Evaluate Dam Safety Programs, Dam Safety Interest Group (DSIG), Project No. T132700-0234

**Sub-Matrix 10: Governance Maturity Matrix**

8 December 2014

**Governance** is defined as the organizational commitment to, and resourcing and oversight of, the effective delivery of a dam safety program and management of dam safety risk.

| Sub-elements | Maturity Level | | | | |
|---|---|---|---|---|---|
| | 1. Needing Development<br>[Lacks conformance to applicable guidelines, standards and industry practice] | 2. Intermediate<br>[Conforms to applicable guidelines, standards and industry practice in some areas] | 3. Good Industry Practice<br>[Generally conforms to applicable guidelines, standards and industry practice] | 4. Best Industry Practice<br>[High degree of understanding and conformance with applicable guidelines, standards and industry practice] | 5. Leading Edge<br>[Developing, trialing and implementing new technology, methods and systems] |
| 10-A. Regulation*<br><br>[Regulation of legislated dam safety requirements, specific to the dam Owner's country or state] | (a) Regulatory requirements are not well recognized and may not be met<br><br>(b) Little or no monitoring of changes in regulatory requirements | (a) Regulatory requirements are recognized and partially met<br><br>(b) Incomplete monitoring of and response to changes in regulatory requirements | (a) Regulatory requirements are generally well understood and met, and satisfactory relationship is maintained with Regulators<br><br>(b) Regular monitoring of and active response to changes in regulatory requirements | (a) Regulatory requirements are well understood and met, and productive relationship is maintained with Regulators<br><br>(b) Proactive monitoring of and high level of engagement with changes in regulatory requirements | Generally meeting Best Practice level, and also developing, trialing and implementing new technology, methods and systems |
| 10-B. Policy, Goals, Values and Risk Management<br><br>[Policy, goals and values that underpin, and provide directive for, delivery of the Dam Safety Program. Risk management policy and practice for the management of dam safety risks.] | (a) Poorly defined or documented with dam safety requirements inadequately addressed, with little or no review and update<br><br>(b) Poor understanding of Dam Safety issues and risks<br><br>(c) Little or no organizational commitment to dam safety awareness<br>(d) Continuous improvement absent | (a) Defined and documented with dam safety requirements partially addressed, with irregular review and update<br><br>(b) Incomplete understanding of Dam Safety issues and risks, and risk management considered generally in isolation from the organization's risk management policy and practice<br><br>(c) Limited organizational commitment to dam safety awareness<br>(d) Continuous improvement is rare | (a) Defined and documented with dam safety requirements addressed and referenced to good industry practice, with regular review and update, and largely integrated with organization policy, goals and values<br><br>(b) Generally complete understanding of Dam Safety issues and risks, and risk management sometimes considered in conjunction with the organization's risk management policy and practice<br><br>(c) Organization is commited to internal dam safety awareness<br>(d) Continuous improvement is present | (a) Well defined and documented with dam safety requirements addressed and referenced to best industry practice, with regular review and update, and fully integrated with organization policy, goals and values<br><br>(b) Comprehensive understanding of Dam Safety issues and risks throughout organization, and risk management fully integrated with the organization's risk management policy and practice<br><br>(c) Organization committed to broader dam safety awareness - internal and external<br>(d) Continuous improvement is embedded | Generally meeting Best Practice level, and also developing, trialing and implementing new technology, methods and systems |
| 10-C. Delegated Roles & Responsibilities<br><br>[Delegation of Roles and Responsibilities for delivery of the Dam Safety Program] | (a) Poorly defined or understood<br><br>(b) Little or no linkage to Dam Safety objectives<br><br>(c) Little or no attention by senior management to dam safety roles and responsibilities | (a) Limited definition and incomplete level of understanding<br><br>(b) Limited linkage to Dam Safety objectives<br><br>(c) Limited review of roles and responsibilities by senior management | (a) Defined and good level of understanding at operational levels in the organization<br><br>(b) Generally linked to Dam Safety objectives, and personnel are empowered to deliver the objectives<br><br>(c) Senior management reviews and confirms roles and responsibilities after organizational changes | (a) Defined, well structured and high level of understanding at all levels in the organization<br><br>(b) Strong linkage to the Dam Safety objectives, and personnel are empowered to deliver the objectives, and influence dam safety outcomes in wider organization<br><br>(c) Senior management regularly reviews roles and responsibilities, including after organizational changes | Generally meeting Best Practice level, and also developing, trialing and implementing new technology, methods and systems |
| 10-D. Internal & External Communication<br><br>[Communications within the organization, and the community, to support delivery of the Dam Safety Program] | (a) Little or no communication between levels and reporting of dam safety issues and risks to Senior and Executive managers<br><br>(b) Little or no provision of dam safety education and awareness information for external communication | (a) Limited communication between levels and reporting of dam safety issues and risks to Senior and Executive managers<br><br>(b) Dam Safety team provides necessary dam safety education and awareness information for communication to external parties | (a) Structured communication between levels and reporting of dam safety issues and risks to Senior and Executive managers with some feedback.<br><br>(b) Dam Safety team involved in external communication of dam safety education and awareness information | (a) No impediments to effective and prompt communication across organization. Reporting of dam safety issues and risks to Senior and Executive managers with feedback.<br><br>(b) High level of Dam Safety team involvement with planning and external communication of dam safety education and awareness information | Generally meeting Best Practice level, and also developing, trialing and implementing new technology, methods and systems |
| 10-E. Resourcing<br><br>[Provision of appropriate personnel and financial resources for delivery of the Dam Safety Program] | (a) Little or no recognition of resourcing needs by organization<br>(b) Dam safety program deliverables are commonly deferred due to lack of resources<br>(c) Little or no consideration of succession requirements | (a) Minimum resourcing needs are met by organization<br>(b) Limited timely completion of key dam safety program deliverables<br>(c) Incomplete consideration and implementation of succession requirements | (a) Resources provided by organization are generally adequate<br>(b) Timely completion of most key dam safety program deliverables<br>(c) Generally complete succession planning and implementation | (a) Organization's dam safety resources provide high value and protection<br>(b) Timely and thorough completion of all key dam safety program deliverables<br>(c) Proactive implementation of succession planning to provide continuity of knowledge and capability | Generally meeting Best Practice level, and also developing, trialing and implementing new technology, methods and systems |

* Evaluate 10-A only if in dam safety regulated jurisdiction

**Figure K1-7: IFT Evaluations of DWR Dam Safety Program Management (shown as highlighted text)**

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# Appendix K2

# General Regulatory Aspects

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## 1.0 BACKGROUND

Dam safety regulation has been in existence literally for millennia. The Babylonian code of law of ancient Mesopotamia (Code of Hammurabi, circa 1754 BC) consists of 282 laws, including some related to the safe operation of dams. It also lays out punishments for contravening these laws, and in the case of dam failure, the available general translation is as follows:

> "53: If anyone be too lazy to keep his dam in proper condition, and does not so keep it; if then the dam break and all the fields be flooded, then shall he in whose dam the break occurred be sold for money, and the money shall replace the corn which he has caused to be ruined." [K2-1]

In more recent times, an Act of British Parliament in 1863 first permitted complaints to be made to magistrates on the safety of reservoirs. The 1868 "Rule in Rylands v Fletcher" was subsequently developed, based on the failure of a dam in 1860 that flooded a neighboring mine, and established dam owners' responsibilities and liabilities. This ruling remains relevant to the present day under Common Law.

In the United States, few states had laws regulating dam safety prior to 1900, even after the 1889 failure of South Fork Dam in Johnstown, Pennsylvania, which caused more than 2,200 deaths. However, in California, the Division of Safety of Dams (DSOD) was formed as a state dam safety regulator under DWR in 1929, following the death of more than 400 people due to the failure of the St. Francis Dam in the previous year. DSOD reviews and approves plans and specifications for both new dams and the alteration and repair of existing dams, oversees their construction, and now annually inspects over 1200 dams in California.

In 1977, following a number of prominent dam failures and incidents, most notably the Teton Dam, Laurel Run Dam, and Kelly Barnes Dam failures, President Carter issued a memorandum directing the review of federal dam safety activities. The first Federal Guidelines for Dam Safety were published in June 1979, and the FERC program for dam safety was also expanded and improved starting about this time.

Oroville Dam is co-regulated by both FERC and DSOD.

## 2.0 FEDERAL ENERGY REGULATORY COMMISSION (FERC)

The predecessor to FERC, the Federal Power Commission (FPC), was created in the 1920 Federal Power Act as a licensing authority for hydroelectric projects in the United States. The FPC was replaced by the Federal Energy Regulatory Commission (FERC) in 1977.

Part 12D inspection requirements for inspection of dams were incorporated into FPC regulations in 1965, which were revised by FERC in 1981 to require new practices and procedures related to reporting safety-related incidents, preparation and implementation of Emergency Action Plans (EAP's), and inspections by independent consultants.

FERC now regulates more than 2,500 hydropower-related non-federal dams across the United States. FERC regulations cover dam design, construction, and operation, and include requirements such as 5-year Part 12D inspections by an external independent consultant and development of EAPs.

In general, FERC regulatory processes have developed over time. The requirement for a Potential Failure Modes Analyses (PFMA, see Appendix F3) was added circa 2002. PFMAs were originally completed over several years in conjunction with 5-year Part 12D reviews. Subsequently, the PFMAs were reviewed, and possibly revised and updated, during the Part 12D inspections. More recently, FERC has required completion of new PFMAs on projects for which it judged that the previous PFMA was not adequate. Also included was the requirement to develop a Supporting Technical Information Document (STID) which is intended to provide a summary of all technical information necessary for the review.

More recently, FERC has requested owners to develop a documented Owners Dam Safety Program (ODSP) with a designated Chief Dam Safety Engineer (see Appendix K1). FERC also introduced the use of Risk-Informed Decision Making (RIDM) as an alternative for dam owners to better prioritize safety-related investigations and upgrades.

## 3.0 DIVISION OF SAFETY OF DAMS (DSOD)

DSOD, although a division within DWR, regulates DWR as a dam owner, as well as other non-federal owners of dams in California. The dam safety regulator and dam-owning agencies being within the same department of state government is not unusual. This is not true for all states, but some other states, such as Colorado and Montana, are organized in this manner.

DSOD does not officially review any aspects of an owner's dam safety program, but remains focused on the technical aspects of the structures themselves. During interviews with the IFT, DSOD personnel indicated that designs are reviewed on a reactive basis within special programs, with an emphasis to proactively prioritize seismic and flood risks.

Typically, studies are initiated as issues come up, or there is a trigger such as the gate failure at Folsom Dam. DSOD has historically had an emphasis on seismic issues, and they have not, until after the Oroville spillway incident, focused on spillways, under the premise that as they have been designed for such large events, spillways should be able to handle smaller flows without difficulty.

A peer review of DSOD [K2-2] was undertaken on behalf of the Association of State Dam Safety Officials (ASDSO) in 2016. The peer review was based on ASDSO peer review guidelines, as well as comparison with the Model State Dam Safety Program [K2-3] developed by FEMA and ASDSO. From the Executive Summary of the peer review report:

> "The team considers the DSOD program to be the *leading dam safety program in the nation*. The statutory authorities of the DSOD meet the minimum requirements outlined in the National Dam Safety Act (NDSA) and most of the recommendations of the Model Dam Safety Program. The senior leadership team is well educated, competent, passionate, committed, and effective, and the DSOD staff is well qualified to execute the dam safety program. The DSOD has a very well documented and rigorous inspection program that is the key component of the surveillance program." [italics added for emphasis]

Although this assessment of the DSOD program is obviously very favorable overall, the peer review does list a number of weaknesses and associated recommendations for general improvement. The following weakness, which are related to DSOD's scope of activities, are relevant to the Oroville Dam spillway incident:

- "The program does not utilize an inspection checklist."
- "At the present time DSOD does not routinely accomplish periodic in-depth analysis to ensure existing dams meet current design standards. The common current reevaluation dam safety industry standard is a 5-year periodic review. Specifically, the re-evaluation of high hazard potential dams every 5 years should include in-depth calculations and evaluations of hydrology, hydraulics, structural stability, earthquake engineering and construction using up-to-date techniques and design criteria."
- "While the DSOD legislation and regulations do not require the detailed re-evaluation of high hazard potential dams every 5 years as described above and as recommended in the model program, the DSOD does in fact perform periodic reevaluations as a matter of policy and practice. The DSOD has a very robust program for periodic reevaluations that is among the best in the nation to ensure that existing dams meet current design standards."

Regarding the last point quoted above, while DSOD was viewed as having a "very robust program for periodic reevaluations," the IFT notes that no such reevaluation was performed for the Oroville spillways, even after spillway erosion concerns were put forward in a 2005 Motion to Intervene (see Appendix C). DWR's response to those emergency spillway concerns was apparently coordinated between DWR and FERC, with DSOD not being involved at all, nor even being copied on DWR's resultant emergency spillway erodibility memo [K2-4]. The IFT also found that the repeated repairs to the service spillway chute were conducted with the awareness of DSOD. However, like most dam safety regulators including FERC, DSOD typically viewed these types of repairs as being "routine maintenance" and therefore generally did not require permit applications for them, nor otherwise closely scrutinize the repairs and their potential significance.

The IFT also notes that, while the the Model State Dam Safety Program [K2-3] cited in the peer review report recommend use of inspection checklists, and included several sample checklists, none of those checklists had sufficient detail regarding spillway defects and failure modes to help identify the spillway failure modes which occurred during the February 2017 incident. To avoid checklists fostering a "check the box" mindset and instilling false confidence, it is essential that checklists be thorough, but not relied upon as a replacement for attentive observation and critical thinking regarding facilities which are being inspected and evaluated.

## 4.0 REGULATORY COORDINATION AND RELATION TO DWR

A figure from Appendix K1 is reproduced at the end of this appendix to illustrate the relationship between DWR, as dam owner, and its two regulators, FERC and DSOD. The figure shows one oddity in the reporting arrangement; FERC officially presents their reports directly to the Director of DWR, whereas the DSOD reports into a Deputy Director within DWR. This is seen by some who were interviewed as an impediment to DSOD's requests being treated with equal weight to those issued by FERC.

Both Regulators require 5-year reviews, and as noted in Appendix F2, separate reviews were held up until 1989. Thereafter, the two reviews have been done simultaneously by a combined team, with reports issued as required to meet both the DSOD and FERC requirements.

The results of the 5-year reviews are reports that identify both required analyses and remediation, which are then undertaken by DWR. The IFT was told that the vast majority of DOE work related to DWR dams is a result of recommendations by the FERC Part 12D Boards. The DWR responses are documented by the Chief Dam Safety Engineer in Accountability Action Reports.

The two regulators attempt to meet quarterly, but at least annually, to ensure coordination. However, as per at least one DWR manager, even if the two regulators agree on a particular issue, one may have additional recommendations which end up further directing DWR efforts.

## 5.0 DSOD RELATIONSHIP WITH DWR AS A DAM OWNER

Although the DSOD is a division within the DWR itself, the IFT found no evidence of any preferential treatment by the DSOD to DWR as an owner, and no interference by DWR as an agency to DSOD's regulation of DWR facilities. Rather, the overwhelming opinion by all who were interviewed was that DSOD was often seen as more stringent on DWR than other owners in the State, perhaps due to the perception of a potential conflict of interest. DSOD personnel indicated to the IFT that DWR had reacted well in the past in responding to large issues. However, other DSOD personnel had a different perspective on the relationship between DSOD and DWR as a dam owner. The IFT heard of DSOD frustration in getting input from DWR, with the feeling that plans sometimes may have been purposefully submitted too late for DSOD to properly influence the decisions. One DSOD interviewee stated that DWR did not work collaboratively "like our other owners," and instead gave DSOD the impression that "they'll deal with it themselves."

Another DSOD interviewee, in describing the February 2017 incident, stated that DSOD personnel felt "pushed aside" during the incident, and were not seen as a regulatory body, but "just as more DWR personnel."

## 6.0 GENERAL COMMENTS – REGULATORY REQUIREMENTS AND PROCESSES

The IFT has provided numerous comments regarding the FERC PFMA process in Appendix F3. In addition, the IFT notes that:

- There does not appear to have been a process to check whether the Supporting Technical Information Documents (STID) actually contained all information required to properly undertake a design review. Rather, in the case of the Oroville STID, it contained a limited selection of drawings and documents, none of which in this case detailed local foundation conditions for either spillway nor details of the service spillway chute slab design.

- FERC apparently relied on the results of the mandatory 5-year review of the Owners Dam Safety Program to judge its efficacy, but in the case of Oroville, this review had not been undertaken as of the Febrary 2017 incident.
- DSOD did not have the resources to review owner's dam safety programs.
- FERC did not have the resources to review dam owner's reports generated from the mandated studies with a team of qualified experts to ensure they were complete and comprehensive with respect to all features being reviewed.

While it was not in the scope of the IFT's investigation to conduct a thorough review of FERC's regulations related to dam safety, in the opinion of the IFT, the Part 12D regulations and other published guidance from FERC have been somewhat ambiguous regarding whether the Part 12D reports were intended to be "updates" on changes since the prior five-year report, versus relatively comprehensive reviews of the facilities which evaluated their design, construction, condition, and performance history relative to current states of practice. Due in part to this ambiguity, the comprehensiveness of Part 12D inspections and reviews has varied in practice, and has tended to be closer to updates of prior reviews rather than independent and comprehensive reviews. The IFT also notes that, like most dam safety regulators including DSOD, FERC typically viewed repairs to spillway concrete chutes as being "routine maintenance" and therefore generally did not require permit applications for them, nor otherwise closely scrutinize the repairs and their potential significance.

The IFT also notes that neither regulator required the development of comprehensive operations, maintenance, and surveillance manuals for dam safety. Such documents are suggested in dam safety guidelines in other countries such as Canada and Australia. From the Canadian guidelines [K2-5]

> "A critical part of the dam safety management system is the development, implementation, and control of procedures for the operation, maintenance, and surveillance of the facility, taking into account public safety and security."

The documents can be used to capture both the "why" that lies behind the operational, maintenance and surveillance requirements, as well as the details of such requirements. From the Australian Guidelines [K2-6]:

> "Its purpose is to ensure adherence to approved operating procedures regardless of the passing of time and changes in operating personnel."

Such a document should capture both the thinking behind, and the details of, the requirement to monitor and understand spillway seepage flows.

As noted above, neither regulator consistently demanded comprehensive design reviews, comparable to the Reclamation SEED process described in Appendix F3, to critically compare older infrastructure to modern best practices for design and construction. This is a very large task, and is well beyond the expectations of 5-year Part 12D inspections and PFMA studies as they were typically conducted in practice, even if, in principle, FERC desired that these reviews be relatively comprehensive. For large dams such as Oroville, such studies would have certainly been beneficial, if done correctly.

It can be argued that regulators should not be responsible for some of the activities discussed above, and that these should be the owner's responsibilities. Regardless, they are activities that need to be undertaken for the sake of public safety, and in the case of Oroville (and in many other situations to the knowledge of the IFT) have not been. Merely meeting regulatory requirements is not sufficient for a dam owner to meet its legal and ethical responsibilities. A culture of doing the minimum to meet regulatory requirements was noted by more than one interviewee. A vital culture change is required.

## 7.0 SUMMARY

Oroville Dam is regulated by two separate bodies, the Federal Energy Regulatory Commission (FERC) and the California Division of Safety of Dams (DSOD). The two regulators are somewhat complementary to each other.

Although DSOD is a division of DWR itself, the IFT found no evidence of any preferential treatment by DSOD as regulator to DWR as owner, and no interference by DWR as an agency onto DSOD as a regulator. However, there are different perspectives on the relationship between DSOD and DWR as a dam owner, and this relationship requires better definition of roles and responsibilities.

The regulatory requirements DSOD and FERC, which are among the leading regulators in the United States, were evidently not sufficient to reveal the potential and actual deficiencies in the Oroville spillways. However, in general, neither regulator has the resources required to fully review dam owner's reports generated from the mandated studies with a team of qualified experts to ensure they are complete and comprehensive with respect to all features being reviewed.

There are also questions regarding the scope of regulatory requirements. Neither regulator did any of the following:

- Mandated a comprehensive design review of the spillways to compare older designs to modern best practices for design and construction. This could have properly informed engineers of the various deficiencies which contributed to the February 2017 incident. While comprehensive reviews may have been the intent of the FERC Part 12D regulations and other guidance from FERC, in practice the Part 12D inspections and reports tended to serve more as updates to the prior reports, rather than comprehensive reviews.
- Reviewed the efficacy of the DWR dam safety program, nor the completeness of available supporting technical information documentation.
- Required comprehensive operations, maintenance and surveillance manuals for dam safety (as recommended in various other countries) which should capture both the thinking behind, and the details of, requirements such as the monitoring of spillway seepage flows.

It can be argued that regulators should not be responsible for these activities, and that these should be the owner's responsibilities. Regardless, they are activities that need to be undertaken for the sake of public safety, and in the case of Oroville (and in many other situations to the knowledge of the IFT) have not been.

## 8.0 REFERENCES

[K2-1] "The Code of Hammurabi". Internet Sacred Text Archive. Evinity Publishing Inc. 2011 (reference as given on Wikipedia).

[K2-2] Peer Review of the Dam Safety Program of the State of California, Department of Water Resources, Division of Safety of Dams, W. B. Bingham, D. J. Mahoney, M. B. Ogden and K. E. Smith, May 19, 2016, Association of State Dam Safety Officials.

[K2-3] Model Dam Safety Program, FEMA 316, July 2007.

[K2-4] Oroville Dam Emergency Spillway, Review of Geologic Conditions, Office Memorandum, F.L. Glick to T. Sutliff, November 29, 2005.

[K2-5] Dam Safety Guidelines, Canadian Dam Association, 2007.

[K2-6] Guidelines on Dam Safety Management, August 2003, Australian National Committee on Large Dams Inc.

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# Appendix L

# Use of the Emergency Spillway

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## 1.0 INTRODUCTION

The final steps leading to the serious erosion downstream from the emergency spillway weir and the evacuation order were the decisions and non-decisions that resulted in the lake level exceeding the emergency spillway weir elevation. The IFT investigated this one aspect of the DWR response to the February 7 slab failure of the service spillway chute, and considers all other aspects of the response to the incident to be outside the scope of its investigation.

The IFT raised the issues associated with the rising lake levels in numerous interviews with those involved from FERC, DSOD, two divisions DWR (O&M and DOE), and the DWR executive level. The IFT also carefully reviewed all available notes taken by DWR in the Incident Command Center from February 8 through February 13. Care has been taken to reproduce the typed and, in some cases, handwritten Incident Command notes verbatim. The IFT has occasionally inserted notes in parenthesis where some explanation is required, and in doing so, has consulted with the note-taker and others to ensure proper interpretation of certain short-form passages. It is noted that these verbatim notes simply reflect conversations within the Incident Command Center at the time, and may not accurately reflect actual field conditions.

## 2.0 BACKGROUND

The timeline of the major events from February 4 through 25, 2017 have been summarized by DWR in a publically-released figure, a portion of which is reproduced as Figure L-1.

Following the February 7 service spillway chute slab failure and closure of the spillway gates, lake levels rose due to ongoing inflows, and it was readily apparent that the service spillway gates would need to be re-opened, or the lake level would rise and overflow the emergency spillway weir. Designed for uncontrolled, free overflow in a major flood event, the emergency spillway had never actually been used. There were a large number of competing concerns in this situation, regarding a number of facilities. These concerns included, but were not necessarily limited to:

- If the service spillway was used:
  - further headcutting, which could endanger both the Pacific Gas and Electric (PG&E) powerlines and eventually the headworks structure itself,
  - almost certain destruction of the entire service spillway chute downstream from the initial failure area,
  - increased tailwater levels and possible flooding of the powerhouse due to partial river blockage,
  - potential for further blockage of the tailrace.
- If the emergency spillway was used:
  - expected erosion of 1 to 4 feet of surficial material, on the basis of the 2009 spillway erosion report, depositing soil, trees and other vegetation into the downstream river,
  - potential loss of transmission towers,

- o unknown potential for additional erosion during first time use, including possible undermining of the emergency spillway weir.

Figure L-2 shows the overall situation prior to the engagement of the Emergency Spillway.

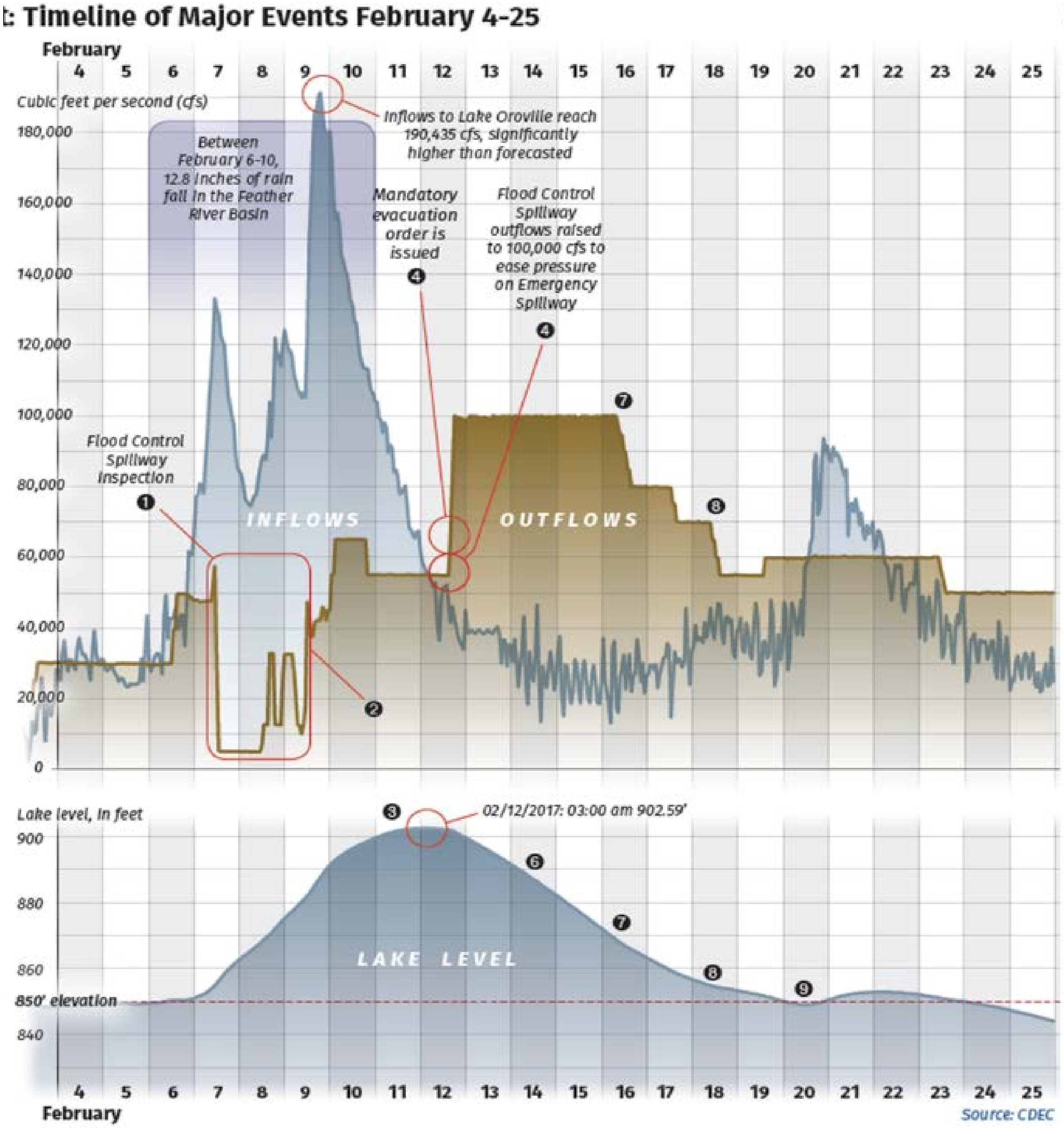


**Figure L-1: Inflows, Outflows and Lake Levels During the Incident [L-1]**


**Figure L-2: Oroville Site during the Incident [L-2]**

The most severe consequences of service spillway use were thought to be the loss of the powerlines and flooding of the powerhouse. Should either of these events occur, flows could not be released through the powerhouse until it was brought back on-line, and without the spillways in operation, lake levels would uncontrollably rise. This could severely curtail the ability to repair the damaged spillway prior to the expected November rains, unless the emergency spillway was used. In addition, downstream water deliveries could be disrupted over the summer once the reservoir level dropped below the sill of the service spillway.

The consequences of using the emergency spillway were less well understood at the start of the incident. However, engineers in the DWR Dam Safety Branch (DSB), DSOD, and DOE, after accessing the geological information for the emergency spillway weir foundation, were very concerned. Early in the incident timeline, they reported the existence of a shear zone under monoliths 18/19, and, after observing the rapid erosion of the service spillway foundation, postulated that similar head-cutting could jeopardize this section of the structure. However, the likelihood of this postulated event was unknown.

Thus, there were two divergent views being presented to the decision-makers:

- One favoring use of the emergency spillway as a means to best reduce the likelihood of the powerhouse going off-line; this view was generally held by Operations personnel and the Executives.

- A second favoring the use of the service spillway; generally held by geologists, dam safety engineers and Incident Command personnel onsite, who recommended increasing service spillway flows to whatever maximum safe flow could be determined on the basis of observing ongoing erosion.

## 3.0 TIMELINE OF THE INCIDENT

### 3.1 <u>Initial Situation</u>

Notes [L-3] from February 8 (11:30 am), the day after the initial slab failure, take stock of the situation:

> "(Executive): If we don't make releases, we are going over the spillway by the weekend…With releases, bottom half of spillway is sacrificial…Will continue to make a mess in the river channel… If we don't release now, we have to double up the flows later. Longer we wait, the more water will have to go down the emergency spillway."

> "(Geology): expressed concerns about emergency spillway. In two hours it (the service spillway) had an intense amount of erosion. Wants to make sure that the rock below the emergency spillway is not the intensely weathered rock. If it is, then it might undermine the emergency spillway."

> "(Geology) wants to make sure that there is not the old rock under the emergency spillway…Is there a fatal flaw in the emergency spillway? Would we find similar erosion under the emergency spillway?"

> "(Geology) Have LIDAR from a year ago where we think water flow will go down spillway. 2-3 million cubic yards of material estimated to hit the diversion pool."

Discussion ensued regarding the amount of flow down the service spillway that could be allowed, and how much erosion was tolerable.

> "(Consultant): try a test flow. (Erosion up to) Station 30 is when you would stop and reevaluate. Use it until you reach station 23."

Essentially, a two-track response was initiated:

- Gates were opened by mid-afternoon for a 20,000 cfs flow release.
- Clearing and grubbing of the area downstream from the emergency spillway was ordered, in case it overtopped.

Meanwhile, Geology Branch forwarded a potential concern for the safety of the emergency overflow weir:

> "17:32 re foundation geologic map from construction under the emergency spillway structure. Most of the foundation is good. 35-75' wide zone under a portion that is intensely to moderately weathered rock similar to the rock under the spillway that was quickly eroded. Concerns: tomorrow follow up …"

At 6:02 pm, the order was given to close the service spillway gates in order to assess the further damage.

> "(Consultant): results are probably as good as we could have hoped for. We knew there would be more erosion. Need to run water not for hours, but for days. Threshold point (for allowable erosion) should be Station 23 in the long haul. Results seem optimistic … We will certainly see more damage, but this is probably the right approach … suggests run 20,000 until the outage tomorrow morning at 0700, then reassess and bump to 25,000. Run for another 24 hours, then possibly ramp up to 30,000."

### 3.2 <u>Changing Hydrologic Conditions</u>

Decision-makers attempted to find the "sweet spot" in balancing the risks of operating the service and emergency spillways: i.e. releasing the minimum flows down the service spillway to limit ongoing erosion, while not overtopping the emergency spillway weir. The decisions had to be made on the basis of hydrological modeling of inflows to the lake, in view of storm forecasts. In view of all the modeling and forecasting uncertainties, this is no easy task at the best of times. In a time of crisis, the decision-making would have been most stressful. As noted previously: "Longer we wait, the more water will have to go down the emergency spillway."

The hydrologic forecast report at 08:30 am the next day (February 9) noted increased inflows:

> "…continue to update modeling runs every 6 hours … Looking at 20,000 spill (assuming 13000 through plant) no longer keeps us out of the emergency spill. Looked at pulsing scenario with 20,000 at night and 35,000 in day – results show … 1 ft below weir. Also looked at constant 35,000 spill keeps us clear through this weekend … assume 09:00 am this morning and continue for next five days."
>
> "If DSOD and FERC and DWR agree that this is a better risk than going over the emergency spillway, that would be the recommendation."

All were in agreement, and the flow was increased to 35,000 cfs by 12:00 pm. However, by the afternoon, the hydrologic forecast was now indicating even higher flows would be required in order not to overtop the emergency spillway weir.

> "12:52 – (Incident commander, O&M managers) determined we will be increasing flows out the Spillway to 50,000 … shutting down the plant … de-energize the high voltage line…"
>
> "13:16 – Executive has made the determination to maintain Spillway release at 35,000."
>
> "15:29 – Thought we had erosion getting close to pg&e line – but it is not as bad as we thought … Wetter forecast … recommend increases down spillway above 35,000 cfs. Need something closer to 50,000 based on latest forecast. That barely keeps us out."

> "18:08 – Current status is that there is 35,000 on the spillway and 7,000 through the plant … Suggestion of combined release rate of 55,000 cfs. Increase to 50,000 on the spillway. 15,000 cfs"
>
> " (Dam Safety) – upstream of breach looks good. Spillway is acting just as it should…The cutback rate appears to have slowed down .…"

By 7:05 pm, the inflow forecast had increased again while the service spillway erosion remained acceptable:

> "calcs show that 55,000 still has us overtopping by Saturday, total releases need to be closer to 70,000"
>
> "PG&E thinks the tower is anchored in good rock"
>
> "(FERC) – erosion uphill seems very slow, erosion towards the tower is also slow… as long as monitoring is flawless, we are ok with increasing the flows. Communications must be spot on."
>
> "Currently bumping flows to 40,000 over spillway, and sending 7000 through Hyatt. DSOD thinks the number should be higher."
>
> "(Dam Safety) – we should be ramping up if the spillway holds. With metrics and monitoring in place, it's not a super high risk to continue to increase flows."

## 3.3 Decision to Accept Emergency Spillway Overflow

Regardless of the opinions that the service spillway erosion was acceptable, a clear decision was made at this point that use of the emergency spillway was to be considered tolerable, and that all efforts would be directed at maintaining the operation of the Hyatt powerhouse:

> "(FERC) – getting uncomfortable with our confidence in how the [service] spillway will perform. Seems like we are dead set (against?) on going an inch over the emergency spillway, but is that really the end of the world?" (the IFT questions the accuracy of this note)
>
> "(Executive) – need to maintain our plant function and protect it at all costs. Want to make sure the powerlines are protected. He does agree that a little water over the spillway might be ok. Concerned the high flows might create issues at the plant … If we lose the towers, it's a long term fix. Don't want to lose the towers … storm has stalled out some, need to prepare the emergency spillway to the best of our availability to take water on Saturday … spend all day tomorrow grubbing [removing the trees and vegetation]. See if we can get that water on the emergency spillway during daylight hours … not as concerned about the 10 or 20,000 over the E/S …"
>
> "(Consultant): it depends on priority. Save tower – or keep water off spillway. I would not go from 35,000 to 70,000 in one step."

> “(DOE): hard to estimate what will happen if we keep adjusting flows. The monitoring will be what drives most of these decisions. We have no way of coming up with a safe flow until after we see what happens with the flow.”
>
> “(Consultant) – want to push past 40,000 cfs. What is monitor time? At least 4 to 6 hours I suggest, If it’s holding at 40,000, at 02:00 are we going to go to 45,000?”

Service spillway flows were ramped up, and by 1:00 am on February 10 outflow was at 55,000 cfs. Outflow was subsequently raised to 65,000 cfs by 3:00 am and maintained at that level as discussion continued through the day whether or not to reduce the flow.

### 3.4 Decision to Reduce Service Spillway Flows

9:00 am February 10 Update call:

> “Erosion is currently stable. Based on current hydrology modeling at 65,000 outflow we won’t need to utilize the Emergency Spillway.”

By 11:00 am, there is significant concern on flooding the powerhouse:

> “Hyatt – flooding in the plant possible … Mitigation; having 2 trucks of sand delivered and 1000 sandbags - and need about 20 crewmembers …”

During the day, the spill quantity versus amount of erosion is discussed in the update calls:

> “Based on 09:00 forecast – if we continue at 65k release, looking at topping out just above 898 at 08:00 tomorrow morning. How long do we need to hold the 65k release? Will be assessing that.”
>
> “Right now, expect that there is hard rock and should be able to continue to spill.”
>
> “How long does Oroville feel like they can pass 65,000 safely? Is there a point in the future when we would cut back?”
>
> “Balance concern for erosion, hydrology, engineering etc. Prepared to have the discussion, they are comfortable with the 65,000”

A conference call at 7:05 pm provides a hydrological update and a major decision:

> “O&M - Bottom line: we continue to show right about 55,000 is the breakpoint ... 65k, if no change we top out at 899.5 at 11:00 am tomorrow. Dialing model in as close as possible. Still some level of risk of a slight possible spill (down emergency spillway) …”
>
> “We’re going to go down to 55k, water ops group and DSOD had some concerns”
>
> “(Consultant) – numbers are within a hair. If we reduce to 55k. Numbers are 900.7, would hit at 20:00 tomorrow night. Could possibly be off 7% of that prediction. O&M recommendation is that we proceed down to 55,000”
>
> “DSOD agree that around 55,000. There might be some trickle over. O&M are planning on reducing to 55,000 after this call.”

> "(FERC) – Monitoring is important to FERC – they are ok with lowering to 55,000."

> The conference call notes end with the observation that:

> "Folks in the room do not disagree."

Service spillway flow was reduced to 55,000 cfs immediately thereafter. From a 7:52 pm meeting:

> "(Executive) –Plant is more important than the emergency spillway in terms of priority. Trusts … opinion from PARO (Power Assessment and Risk Office). He is on the same page to save the plant before anything else."

## 3.5 Emergency Spillway Weir Overflow and Subsequent Events

At some point it became apparent that the emergency spillway would come into play unless service spillway releases were increased. During the evening of February 10 and through the early morning of February 11, preparations continued to clear all personnel and equipment from the areas downstream from the emergency spillway weir. Spills from the service spillway were kept at 55,000 cfs, although models were being run to see the effects of increasing spills to either 65,000 or 75,000 cfs.

Emergency weir overflow commenced sometime between 7:00 and 8:00 am:

> "07:00: 3,300 over aux (emergency) spill ; calling (Executive) can we bump back up to 60,000"

> "07:38:will get there today … inflow vs spillway – differential builds head. Eventually it will go over … 10,000 cfs Aux spill for up to 2 days, think it will be 1 day"

> "07:48 keep spills at 55k. Who's call to open the spillway more? Powerline is more important than Aux (Emergency) spillway"

> "07:55 (Incident Commander) – going over E/S"

Service Spillway flows continued at 55,000 cfs through February 11 and into the next day (February 12), when at 1:53 pm:

> "Large section of left side of horseshoe shaped area [immediately downstream from the emergency spillway weir] eroded suddenly. Could impact tailrace"

> "14:57 Geology is saying that we need to increase flows down the gated spillway to relieve pressure on the AUX … Management … says maintain flows. Afraid they will flood the plant. Tailrace at 242.86 @ 14:00"

> "15:25… started headcutting in Monolith 3; could reach in 2-4 hours"

> "15:35…decision has been made to spill to 65k"

> "15:44 Start evacuation"

Spills were subsequently increased to greater than 90,000 cfs, and reached 100,000 cfs . By 5:48 pm it was evident that additional headcutting at the service spillway was not going to be an issue:

> "Appear there was no increased damage to spillway. Appears erosion is holding steady if not looking promising. All staff evacuated. Minimal crew on spill deck: 2 FERC, 1 DOE sup. turning on lights"

The main concern was then focused on the rising tailwater level and the potential for Hyatt powerhouse flooding. By 7:42 am on February 13:

> "Tailrace elevation is 6" from flooding the plant. Was raising (sic) – then stabilized."

By this time, the sandbag berms around the stoplog hatch covers in the powerhouse had been reinforced, as shown in Figure L-3. Additional measures to seal the covers were also put in place. The tailrace level stabilized more than 2 feet above the powerhouse flood level, while the 100,000 cfs service spillway flow continued until midway through February 16. The Hyatt powerhouse did not flood due to, in the words of one interviewee, " the heroic efforts of the crew using jury-rigged sealed covers designed by O&M staff."

**Figure L-3: Sandbags in Hyatt Powerhouse [L-4]**

## 4.0 IFT COMMENTS ON THE INCIDENT

The IFT recognizes the difficulty of the situation and the many unknowns with which those on site during the incident had to deal. Rather than criticize the hour-by-hour decisions that had to be made given these conditions, the purpose of the following discussions is to investigate what factors led to the decisions, and whether any changes could improve such decision-making in the future.

In hindsight, the incident is a story of "too little too late," in regard to releasing flows down the service spillway. Every delay in increasing flows meant even higher flows were necessary later on to keep the lake level below the elevation of the emergency spillway weir crest. Had the advice of civil engineers and geologists been taken over that of electrical and mechanical engineers, service spillway flows would have been increased to the maximum possible without inflicting damage to the transmission tower or the powerhouse. With this observational approach, and in hindsight, the emergency spillway crest would likely not have overtopped, and the evacuation of the downstream public would have been avoided. But, as one Executive expressed during interviews, it "hurt everyone in the organization to see what happened, and the situation had nothing to do with lack of caring."

This is not a matter of placing blame; the main question is whether or not disparate opinions were given appropriate weight in the decision-making process, and, if not, what needs to be changed going forward to ensure an appropriate balance between conflicting requirements is achieved (refer to Appendix K1 for comments on corporate dam safety culture).

### 4.1 Decisions to Delay and Reduce Service Spillway Flows

As seen in the above Incident Command notes, there were two critical decision points that delayed releasing flows down the damaged service spillway, and essentially guaranteed that the emergency spillway would be overtopped, at least by a few inches:

- 1:16 pm on February 9 - the decision not to increase flows immediately from 35,000 to 50,000 cfs, but rather "ramp up" more slowly. Flows more than 35,000 cfs were not allowed until about 6 hours later, and flows more than 45,000 cfs were not reached until about 11 hours later.
- 7:05 pm call on February 10 - the decision to reduce flows from 65,000 to 55,000 cfs, and then holding to this decision until just before the evacuation order some 44.5 hours later.

These decisions likely resulted in a least a few feet of reservoir rise during the incident, based on approximate calculations by the IFT.

The first decision (1:16 pm February 9) led to a significant delay in increasing spillway flows beyond 35,000 cfs, and was made although all indications in the notes are that the service spillway erosion at the time was quite tolerable.

The second decision (7:05 pm February 10), as per one interview, took those in the room by surprise, in view of the stable erosion situation and the lack of a safety buffer on the reservoir level estimates. The IFT was informed by more than one interviewee that the double-negative "Folks in the room do not disagree" was written with intended purpose; there was silence rather than explicit agreement from most, and those most previously vocal in their opposition were not part of the decision, including the Chief Dam Safety Officer (CDSE). The IFT heard the same double negative from interviews with DSOD personnel. The decision was remembered by some DWR personnel on site as being "inexcusable," and there were efforts to have it overturned. As per Incident Command notes [L-2], these on site personnel were told:

> "(Executive) made the decision hours ago – nothing we can do to make any change at this point."

The decision to maintain the 55,000 cfs until the evacuation order was given, while erosion in the emergency spillway continued, is also recorded. From the 6:00 am briefing on the morning of February 13:

> "(Incident Commander) wanted to increase flows, mgmt. said to hold, (IC) did, regrettably, and we lost time because of it."

### 4.2 Drivers for the Decisions

The IFT heard a number of different viewpoints regarding the drivers behind these decisions. The record clearly indicates that DWR Executives considered mitigating the risk of powerhouse flooding to be paramount, and this indication was repeated by others in a number of interviews. The other main driver was the potential for further headcutting up the service spillway, which would in turn endanger the transmission tower and possibly even the service spillway headgate structure. Thus, two factors were necessarily being considered:

- Tailwater level as compared to powerhouse flooding level.
- Ongoing erosion at the service spillway.

The IFT investigated the actual conditions associated with each of these factors at the time of the decisions.

**Tailwater Rise –**To better understand the potential for powerhouse flooding that would have influenced the two critical decisions, the IFT prepared Figure L-4 from information provided by DWR.

The annotated graph shows tailwater response due to service spillway gate operations. Tailwater first drops when the gates are closed soon after the service spillway chute failure to allow inspection of the initial failed area, and then rises when the gates are re-opened for a number of trial flows. The first decision, overruling the Incident Command to increase flow beyond 35,000 cfs, was made at a time when powerhouse flooding was not a serious threat. However flooding of the River Valve Outlet System (RVOS) was imminent, and the potential for loss of the transmission tower had already been identified:

> "19:05 Feb 9 (Consultant): it depends on priority. Save tower – or keep water off spillway."

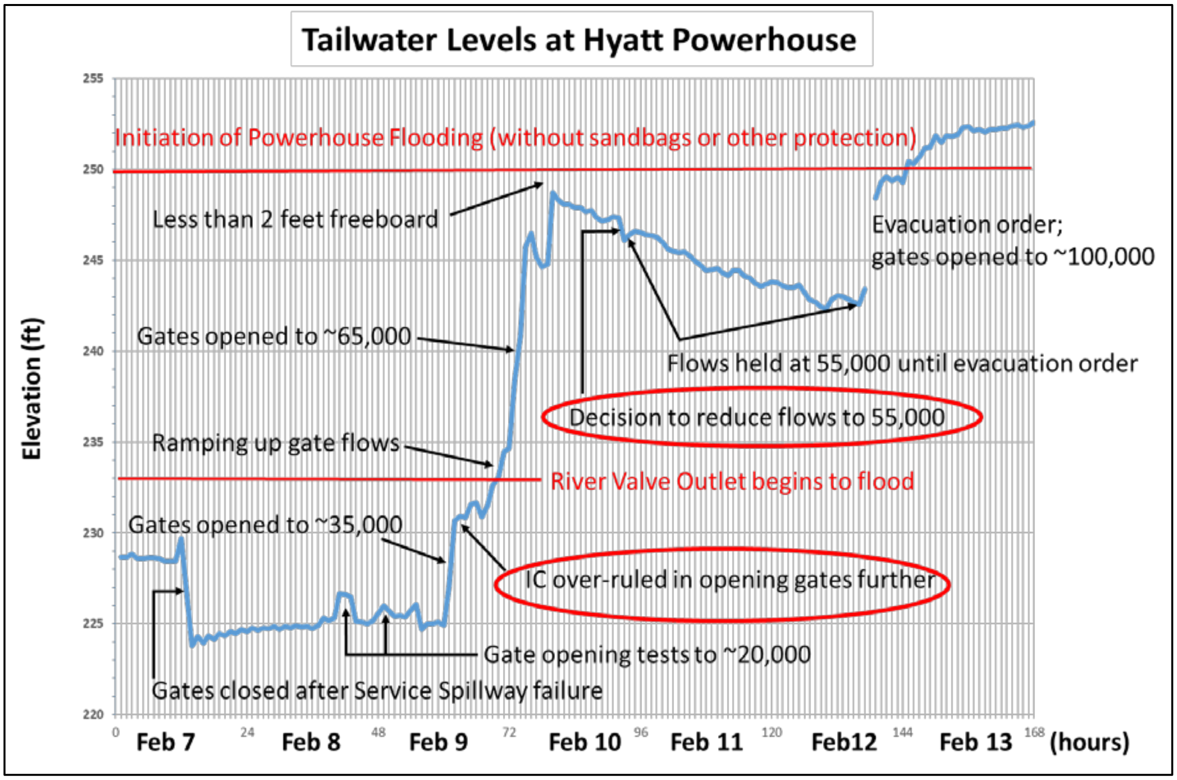


**Figure L-4: Tailwater Levels During the Oroville Incident**

The RVOS had the ability to pass about 4,000 cfs (in its condition at the time of the incident) in addition to the powerhouse flows, although it draws cooler water from lower in the reservoir, and its use is usually reserved for late summer to assist in fish stocks. It could have been used to somewhat reduce flows down the service spillway. It could also have been used to ensure some continuity of flow out of the reservoir if the powerhouse had flooded. However, the system remained closed during the incident.

The RVOS was flooded soon after the delayed decision was taken to increase service spillway flows beyond 35,000 cfs. Once the spill was eventually increased to 65,000 cfs, Figure L-4 shows that tailwater elevation peaked less than 2 ft below the powerhouse flooding level (without protection). Tailwater then steadily receded over the next 11 hours, before the second critical decision (to reduce flows to 55,000 cfs) was made. Thus, the powerhouse was again not in imminent danger of flooding at the time the decision was made to reduce service spillway discharge. As tailwater was being closely monitored and reported, this fact should have been known by the decision makers. Tailwater then continued to drop over the next 24+ hours, until the evacuation order became necessary. However, the observational method that had previously allowed spillway flows to be increased was not reinstated.

Incident Command notes of Executive statements indicate that powerhouse flooding is the apparent driver behind these decisions. However, the IFT concludes that tailwater rise, and the possibility of powerhouse flooding, was of no imminent concern at the time of the first decision, and of diminishing concern at the time of the second decision.

**Further Erosion at the Service Spillway –** The other driver is the continued erosion of the service spillway. From notes taken at 7:05 pm on February 9:

> "(Consultant): it depends on priority. Save tower – or keep water off spillway"

FERC personnel confirmed in interviews that their main concern at the time was the transmission tower. But the IFT heard a contradictory view during DSOD interviews, in which it was stated that although there was some ground cracking around the tower, ongoing monitoring showed that it was not moving, and that the engineers believed the tower was stable in the condition at the time. There was the possibility that further headcutting could put the tower in danger, but engineers in DSB, DSOD and DOE believed that head-cutting back to the service spillway headworks would not occur, on the basis of their geological review, which showed significantly better rock conditions upstream from the initially failed area.

The IFT further concludes that more erosion along the service spillway, although a very real and obvious threat, was not an immediate and pressing concern at the time of the decisions. The question is why there was no effort to continue to follow an observational approach and allow service spillway releases to be increased, or at least, be maintained.

## 4.3 Understanding Consequences in the Risk Tradeoff

The IFT was told that the risk of tailwater rise due to erosional debris from potential erosion downstream from the emergency spillway was equivalent to that with the continued use of the service spillway. With these risks being treated as equal, the risk tradeoff involved the potential loss of the powerlines and flooding of the powerhouse, with significant issues with control of lake level and downstream water deliveries until the powerhouse could be brought back on-line, versus the unknown risk of using the emergency spillway. However, the IFT believes (through multiple interviews) that neither the probabilities nor the consequences of these disparate risks had been adequately reviewed and laid out for the decision makers. There were many opinions based on each individual's perspective, but little fact. Those with electrical/mechanical backgrounds apparently stressed the consequences of powerhouse or transmission line losses; those with a civil/geotechnical background were adamant that the risks of using the emergency spillway should trump all else, without regard to the potential for powerhouse flooding and the associated consequences. The IFT considered these consequences in turn.

**Loss of Transmission Tower beside Service Spillway –** The duration that the powerhouse could possibly be off-line after flooding was not known, and estimates varied widely, from days to months. FERC personnel had been told there would be a six-month outage if the tower was lost (from interviews). The IFT queried the Vice-President of Field Operations at another utility that has also been faced with the potential loss of a major transmission tower, to test this assumption. It was noted that this utility had developed a number of contingency plans for the loss of a major tower, in order to have any transmission line back in service within days to two weeks, including:

- Spare towers available in Operations yards to cover both geographical extent of the service area and tower type.
- Plans to use training towers (i.e. moving towers from training facilities).

- Temporary use of cranes with 3-phase lifts on insulated booms.
- Removal of the threatened tower, allowing line sag close to ground, and establishing emergency safe limits of approach.

In interviews with DWR Executives, the IFT learned that such contingency planning was lacking at DWR. Although it was thought that there were no spare towers available, it was learned only after the incident that a suitable replacement tower was available at a powerline training facility in California. The IFT concludes that the consequence of the loss of the tower was not properly evaluated during the decision making process.

**Erosion Downstream of Emergency Spillway –** The potential consequences of allowing the emergency spillway to activate had been identified early in the incident. From February 8 notes [L-3]:

> "(PG&E is) convinced they will lose the (transmission) tower if we spill over the emergency spillway."
>
> As such, DWR targeted specific areas for clearing and grubbing, specifically to protect the PG&E towers. Erosion downstream of the emergency spillway was also being envisaged as a major issue by many of the technical staff. One DOE person involved with the incident stated that he "envisioned the same amount of erosion from the hillside downstream from the emergency spillway as had been seen at the service spillway."

DSOD personnel gave the IFT similar viewpoints. Although not aware of the 2005 and 2009 erosion studies at the time, following a quick review of documents including borehole logs from predesign and design explorations, they completed a geologic interpretation of the amount of expected downcutting based on the weathering profile. They told the IFT that they were not at all surprised at the erosion once the emergency spillway started to overflow. It is unclear however, whether this message was ever fully relayed to the Executives. The DSOD personnel also "fully expected" the service spillway erosion not to proceed further upstream. From a 2:00 pm call on February 10:

> "Right now, expect that there is hard rock and should be able to continue to spill"

DOE personnel also expressed the opinion to the IFT that, as Subject Matter Experts, they thought the safer place to have the erosion was at the service spillway. This was based on a judgment of upstream rock conditions that would not allow erosion to progress to the headworks structure.

**Flooding of the RVOS –** Interestingly, the potential for, and the eventual flooding of the River Valve Outlet System (RVOS) is not mentioned in the Incident Command notes, nor was this issue raised in any of the IFT interviews. Although loss of the RVOS would eliminate a redundant means to pass flows out of the reservoir should the powerhouse be flooded, the IFT surmises that this issue was not fully considered for two possible reasons:

- Even if both the powerhouse and the river outlet valve itself were to flood, emergency controls for the RVOS are located well above any possible tailwater elevation (290 ft, 40 ft above the powerhouse flooding level) [L-5], so that there could have been limited concern regarding flooding of the valve itself.

- As per interviews, there was still a general feeling that the situation could be controlled, and there was no acceptance that the powerhouse was going to be flooded. As such, there was no preparation to allow for this alternative means of passing flows, that could have been critical were the powerhouse to flood, as next discussed.

**Flooding of the Powerhouse and Loss of Downstream Water Conveyance –** Flooding of the powerhouse would certainly result in the powerhouse being off-line for months, with very limited capacity to pass flows downstream from Lake Oroville. As such, there could be issues in regard to safe water management during the remainder of the winter and during the necessary service spillway repairs. In addition, there would be limited downstream water availability in the coming summer, once reservoir levels were below the spillway sill elevation, affecting both water deliveries and the environment. In one interview with an Executive, the limited water delivery was deemed as potentially one of the biggest disasters in the history of California.

The powerhouse would be expected to pass up to about 14,000 cfs downstream; if offline, the maximum downstream flow (without use of the spillways) would be limited to 4,000 cfs using the River Valve Outlet System (RVOS), assuming emergency river valve controls (well above powerhouse flood level) would remain available. There were three considerations from a water management perspective:

- **Flood control releases:** Powerhouse releases could have been necessary to augment service spillway capability for managing high reservoir inflows through the remainder of winter and spring. Although shown to be performing adequately at 55,000 cfs flow, uncertainty existed whether this situation would remain stable through operation for the remainder of the winter. Should the service spillway and the powerhouse both become fully inoperable, there could have been uncontrolled releases over the emergency spillway for extended periods of time throughout the spring and early summer.

- **Water deliveries**: Restricting releases from Lake Oroville would impact downstream pumping levels for the State Water Project Contractors. However, during drought years, water allocations in the past have dropped to near zero. The actual water flows in the Sacramento-San Joaquin Delta (Delta) also depend on inflows from other sources, including releases from the Shasta and Folsom reservoirs by the Bureau of Reclamation. Water availability for Southern California also depends on the San Luis reservoir south of the Delta. Since 2017 was a very wet year, it was opined by DWR personnel that water release restrictions at Oroville over one summer would not have been as dire for south of Delta, as during the recent drought years.

- **Environmental concerns**: Minimum instream flow for environmental purposes on the Feather River is set at 1,700cfs in wet years, reducing to 1,000 cfs (nominal) in summer. Peak deliveries to satisfy local settlement contractors are greater than 3,000 cfs in the late spring and summer. Thus, arguably, if the RVOS remained operable, releases would have been enough to meet these minimum requirements. However, there are also obligations for

> outflow and water quality objectives in the Delta in addition to providing a “freshwater corridor” for water supply deliveries. These requirements are in addition to the minimum instream release requirements. There are also temperature considerations – colder, lower reservoir water is conserved for release from Lake Oroville later in the summer and fall to aid fish stocks, but by using the RVOS, this colder water may have been exhausted earlier in the year. In general, had water releases been restricted, DWR may have had to request significant deviations from water quality and temperature standard requirements, and real world environmental consequences could have ensued.

The IFT concludes that, beyond simply damaging the electrical/mechanical equipment itself, powerhouse flooding could have had very serious environmental consequences because of the reduction in water quality due to the restricted flows, and rise in water temperature in late summer/fall due to lack of control over the depths of water draws from Lake Oroville. The lack of release capability through the powerhouse would also have required the damaged service spillway to be used longer into the 2017 season, delaying the start of the spillway reconstruction work.

Rather than being portrayed as potentially one of the biggest disasters in California history, the reduction in water availability to downstream Contractors would have perhaps been more correctly portrayed as presenting significant business and legal challenges, but actual reductions in water deliveries would have been no worse than in the drought years.

**Undermining of the Emergency Spillway Weir –** The potential for serious erosion downstream from the emergency spillway weir, and of undermining the weir itself was raised very early in Incident Command Center conversations (early evening of February 8). However, in numerous interviews with various parties, the IFT cannot find evidence that anyone truly expected the amount of erosion that actually occurred, or if they did, that this was raised during the decision making process. This is explored further in the following section.

### 4.4 Dynamics of Decision-Making:

**The Decision-Maker –** It was documented early-on who was to make the decision regarding further use of the service spillway. From the February 8 Incident Command notes [L-3]:

> “Who is making the call? DWR director, DSOD and FERC.”

However in later interviews, FERC personnel informed the IFT that in no way was FERC a decision-maker – FERC was there in a support role, only to “step in” if necessary.

DSOD regulatory personnel equally denied being a decision-maker in the Incident as well, and later notes are clear that the DWR Executives made the final decisions in the delays and reductions of service spillway releases.

In answer to the IFT interview question “Were the different potential consequences being properly described and ‘weighted’ for those who had to make the decisions?” interviewees stated simply that it was the Director’s call.

**Supporting Staff and Lines of Communication –** Within DWR, interviews clearly identified that O&M electrical/mechanical personnel and DOE civil/geotechnical personnel held very different positions during the incident. DOE personnel indicated that they participated in all the calls, but were not part of the Incident Command Team, and not directly involved with decision making.

Although part of O&M, and also taking part in the regular incident updates and phone calls, the IFT was told that at no time during the Incident did the Director of DWR have any direct contact with the Chief Dam Safety Engineer.

Some DSOD personnel also noted that they felt ignored during the incident, and needed to stay with FERC onsite so as to "keep in the loop." Interestingly, one DWR Executive involved with the incident opined in an interview that it was his desire to have DSOD more closely involved with the decision making process, but that this was not supported by others in the DOE and O&M Divisions.

The degree of separation between DWR and DSOD was not recognized by many involved, who thought DSOD personnel were "just other DWR employees trying to have a say." The DSOD personnel indicated that all information was shared with FERC on site, but they did not know whether this information was transferred to the DWR Executives. They noted that DOE and DSB personnel raised their concerns through their organizational channels and provided advice on what they considered to be safe flows for the damaged service spillway, in order to try to not use the emergency spillway.

The IFT was told in one response to an IFT survey that dam safety management decisions were being made over the phone by executive level employees who did not have the "boots on the ground' situational awareness. From various interviews, there was a general feeling that the Executives were dismissive of the technical concerns being raised by civil engineers and geologists.

DOE personnel expressed the opinion that, although being vehemently opposed to using the emergency spillway, only a few people "had the ear" of the Executives, and that they eventually agreed to let the emergency spillway overflow "by a few inches."

In a number of interviews, the IFT noted that some personnel blamed themselves for not having the courage to demand increased outflow in the service spillway, and that, as the decisions were being made "at the top," they "backed off."

## 4.5 Reliance on Hydrologic Modeling

Although there were obviously efforts to avoid using the emergency spillway, not everything was done to stop it, and, rather, there was an effort to find the "sweet spot," which ultimately failed.

Once the emergency spillway was being overtopped, it was noted on February 11 at 10:19 am:

> "We knew the aux spillway was going to be used … (Executive) – we thought the recession curve (of the hydrograph) was more steep. Models were wrong."

In fact, the models were not necessarily wrong, the outcome was well within the uncertainty bounds. The IFT was informed that the inflow forecast model had been shown to be fairly accurate during the first few days of the incident. It is well known that any such model, however seemingly accurate in the short term, still has a wide band of uncertainty around a predicted mean or median outcome. The IFT was told that there was a conscious decision during the incident to rely on the "deterministic best value" (i.e. the median), although typically a full suite of outcomes would be considered under normal operations. As previously noted, the reasoning given for this was that the risks of further service spillway use versus emergency spillway activation were estimated be about equal in terms of the threats to downstream water conveyance. There was also the belief (at least by the decision-makers) that a relatively small amount of flow over the emergency spillway would be acceptable. Because of this perceived balance of risks, the median inflow forecasts (absent any hedging factors) were used for reservoir projections.

As the incident progressed, the Incident Command notes indicate attempts at "dialing (the) model in as close as possible" with "numbers…within a hair" in determining that "there might be some trickle over" the emergency spillway weir. Some minor uncertainty was acknowledged, "Could possibly be off 7% of that prediction," but the perceived balance of risk was not questioned further, and the consequences of being off the projected deterministic best value of the model were apparently not discussed. The decision not to account for a reasonable band of uncertainty led to strong, but misplaced, confidence that, at worst, the amount of emergency spillway weir overtopping would be minimal.

## 5.0 SUMMARY

The final steps leading to the serious erosion downstream from the emergency spillway weir and the evacuation order, were the decisions and non-decisions that resulted in the lake level exceeding the emergency spillway weir elevation. The IFT investigated this one aspect of the DWR response to the February 7 service chute slab failure through numerous interviews and careful review of available notes taken by DWR in the Incident Command Center. The IFT recognizes the difficulty of the situation and the many unknown with which those on site during the incident had to deal. Rather than criticize the hour-by-hour decisions that had to be made given these conditions, the purpose is to investigate what factors led to the decisions, and whether any changes could improve such decision making in the future.

Following the February 7 service spillway chute slab failure and closure of the spillway gates, lake levels rose due to ongoing inflows, and it was readily apparent that the spillway gates would need to be re-opened, or the lake level would rise and overflow the emergency spillway weir. Either option presented risk. Continued use of the service spillway could result in further erosion, which could endanger the powerlines or the powerhouse due to rising tailwater levels. Should either the powerlines be lost or the powerhouse flooded, flows could not be released through the powerhouse until it was brought back on-line, and without the spillways in operation, lake levels would not be controllable. This would also severely curtail downstream water deliveries and affect water quality in the coming summer, once the reservoir level fell below the service spillway gate sill elevation. In addition, there could be complications in water management during repair of the damaged spillway prior to the expected November rains, unless the emergency spillway was used. The duration that the powerhouse could possibly be off-line was not known, but estimates of up to six months or more were given under either scenario, loss of powerlines or powerhouse flooding.

Allowing first-time use of the emergency spillway could also result in significant downstream erosion, leading to downstream channel blockage and powerhouse flooding, as well as possible headcutting back up to the overflow weir. Although the 2009 Emergency Spillway Erosion report (see Appendix C) had stated only minimal erosion would be expected, engineers in DSB, DSOD and DOE, after accessing the geological information for the emergency spillway weir foundation, were very concerned. They identified a shear zone under monoliths 18/19, and in observing the rapid erosion of the service spillway foundation, postulated that similar head-cutting could jeopardize this section of the emergency spillway structure. These engineers also believed that head-cutting back to the service spillway headworks would *not* occur, based on the available geological mapping, which showed significantly better rock conditions upstream from the initially failed area.

There were thus two divergent views being presented to the decision-makers:

- One favoring use of the emergency spillway as the best means to reduce the likelihood of the powerhouse going off-line; this position was generally held by Operations personnel and the DWR Executives.
- The other favoring the use of the service spillway; this position was generally held by geologists, dam safety engineers and Incident Command personnel onsite. These persons recommended increasing service spillway flows to whatever maximum safe flow could be determined on the basis of observing ongoing erosion; an observational approach.

Decision-makers attempted to find the “sweet spot” in balancing the risks of operating the service and emergency spillways: i.e. releasing limited flows down the service spillway to both reduce ongoing erosion and prevent powerhouse flooding, while not overtopping the emergency spillway weir. The decisions had to be made on the basis of hydrological modeling of inflows to the lake, in view of upcoming storm patterns, and updated forecasting, which progressively indicated that greater service spillway flows would be required in order not to overtop the emergency spillway weir. However, the modelling continued to indicate that the maximum lake level would be “within a hair” of overtopping the emergency spillway, and there may be only “a dribble over.” The situation was summarized in the Incident Command notes as “Longer we wait, the more water will have to go down the emergency spillway.”

During this time of risk-tradeoff assessments, there were two critical decision points that delayed releasing flows down the damaged service spillway, and essentially guaranteed that the emergency spillway would be overtopped, at least by a few inches:

- 1:16 pm on February 9 - the decision that overruled the Incident Command and delayed increasing flows beyond 35,000 cfs for about 6 hours.
- 7:05 pm call on February 10 - the decision to reduce flows from 65,000 to 55,000 cfs, and then holding to this decision until just before the evacuation order some 44.5 hours later.

Although Incident Command notes of Executive statements indicate that powerhouse flooding is the apparent driver behind the decisions, the IFT reviewed tailwater records from the incident and concluded that the powerhouse was not being seriously threatened by flooding at the times when these decisions were made. At the time of the first decision the tailwater level was about 20 feet below the flood level at the powerhouse. Subsequently, the tailwater peaked less than 2 ft below the flood elevation (sandbags had been put in place but were not required). The tailrace level then steadily receded over the next 11 hours, until the second critical decision, to reduce flows to 55,000 cfs, was made. Tailwater then continually dropped over the next 24+ hours until the evacuation order became necessary.

The remaining driver is the continued erosion of the service spillway. However, all indications in the notes are that the erosion at the times of the two decisions noted above was quite tolerable, and that no assets were in any immediate danger.

In the end, the decision-makers decided, although records show that tailrace levels were dropping and service spillway erosion was stable, to limit service spillway discharge to further reduce the likelihood of powerhouse flooding. This gain in margin of safety against the risk of powerhouse or transmission line loss was at the expense of the risk of using the emergency spillway, with essentially no margin of safety. In limiting service spillway discharge, the additional dam safety risk associated with use of the emergency spillway was not appropriately considered. Once the emergency spillway was allowed to overtop, this additional risk was soon realized, and the evacuation order became a necessary precaution.

The decisions to abandon the observational approach and limit service spillway discharge were made against the advice of Incident Command personnel on site, and subject matter experts in both DSOD and DOE. Why was this? Through Incident Command notes and various interviews, the IFT believes that there were a number of reasons, including:

- **Understanding of Consequences –** The IFT believes (through multiple interviews) that neither the probabilities nor the consequences of the two disparate risks were ever adequately reviewed and laid out for the decision makers. There were many opinions based on each individual’s perspective, but little fact. Within DWR, interviews clearly identified that O&M electrical/mechanical personnel and DOE civil/geotechnical personnel held very different positions during the incident. Those with electrical/mechanical backgrounds apparently stressed the consequences of powerhouse or transmission line losses; while those with a civil/geotechnical background were adamant that the risks of using the emergency spillway should trump all else, without regard to the consequences of potential loss of significant downstream flow should the powerhouse stay offline for some time.
- **Reliance on Hydrologic Forecasting and Modeling –** Model uncertainty, although acknowledged, was intentionally ignored as the incident progressed, as DWR was “dialing (the) model in as close as possible” with “numbers … within a hair” in judging that “there might be some trickle over” the emergency spillway weir. The use of mean model values was based on the belief that the risks of powerhouse flooding due to either the continued use of the service spillway or the use of the emergency spillway were essentially equivalent. Although perhaps true in regard to powerhouse flooding, the inherent additional risks associated with using the emergency spillway were not being accounted for.
- **Dynamics of Decision Making –** Interviews and notes make it abundantly clear that nearly all IC, DOE, and DSB personnel were at odds with the two major decisions, but these personnel, as well as DSOD and FERC personnel, rather than openly agreeing, “did not disagree” with the DWR Executives. Although the potential for serious erosion

downstream from the emergency spillway weir, and of undermining the weir itself was raised very early in Incident Command Center conversations, the IFT cannot find evidence that anyone truly expected the amount of erosion that actually occurred in the emergency spillway discharge channel, or, if they did, that this was raised during the decision making process. In a number of interviews, the IFT noted that some personnel blamed themselves for not having the courage to demand increased outflow in the service spillway.

The incident is a story of "too little too late" in regard to releasing flows through the service spillway. Every delay in increasing flows meant even higher flows were necessary later on to keep the lake level below the elevation of the emergency spillway weir. In hindsight, had the advice of civil engineers and geologists been taken over that of electrical/mechanical engineers, service spillway flows would have been increased to the maximum possible extent without inflicting damage to the transmission tower or the powerhouse, based on an observational approach. With this approach, the emergency spillway would most likely not have overtopped, and the evacuation of the downstream public would have been avoided. The IFT believes that all decisions were made with the best of intentions, but that the risk tradeoffs were not adequately assessed, and there was an unequal balance of uncertainties applied to equally serious potential outcomes.

What could be different? The IFT believes that the dynamics of the decision-making and the unequal weighting between the risks of continued service spillway use and emergency spillway weir overflow are both a direct reflection of the relative immaturity of the dam safety program within DWR (refer to Appendix K1). The decision-makers were not in a position, or did not have the experience and background, to adequately understand the dam safety uncertainties underlying their decisions. With a more mature and "top-down" dam safety program, a well-informed senior executive with the pre-defined overall responsibility for dam safety would not be unduly influenced by one point of view, and would be in a much better position to make the necessary risk tradeoffs in critical decisions, such as those that were required during the Oroville Dam spillway incident.

## 6.0 REFERENCES

[L-1] Lake Oroville Spillway Incident: Timeline of Major Events February 4-25, DWR, http://www.water.ca.gov/oroville-spillway/pdf/2017/Lake%20Oroville%20events%20timeline.pdf

[L-2] Photo provided by DWR

[L-3] Incident Command Notes provided by DWR : typed notes from February 8 – 12: handwritten notes from February 12 and 13, 2017

[L-4] Photo provided by DWR, dated February 13, 2017.

[L-5] Chapter 5, California State Water Project Bulletin 200, California Department of Water Resources, November 1974

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# Appendix M

# Resumes of Independent Forensic Team Members

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## LIST OF RESUMES OF INDEPENDENT FORENSIC TEAM MEMBERS

John W. France, PE, D.GE, D.WRE – Team Leader and Geotechnical Engineer

Irfan A. Alvi, PE – Hydraulic Structures Engineer and Human Factors Specialist

Peter A. Dickson, PhD, PG – Engineering Geologist

Henry T. Falvey, Dr.-Ing, Hon.D.WRE – Hydraulic Engineer

Stephen J. Rigbey – Director, Dam Safety at BC Hydro, and Geological Engineer

John Trojanowski, PE – Hydraulic Structures Engineer

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# John W. France, PE, D.GE, D.WRE

### Professional History

Years with AECOM (Legacy URS): 25
Years with other firms: 16

### Academic Training

MS, Civil Engineering, Cornell University, 1976
BS, Civil Engineering, Cornell University, 1972

### Areas of Expertise

Dams and Dam Safety
Geotechnical Engineering

### Registration

Professional Engineer: CO, OK, MA, NC

Mr. France has more than 41 years of experience in engineering consulting and design. Most of Mr. France's technical work for the past 33 years has focused on dams and water retention structures. This experience includes dam safety inspections and analyses, detailed geotechnical and geological field and laboratory investigations, hazard classification, seepage and static stability analyses and evaluations, seismic stability/seismic deformation analyses, conceptual and final designs of new structures, rehabilitation of existing structures, and consultation during construction. He has served on numerous senior technical review boards / panels for the U.S. Army Corps of Engineers (USACE); the U.S. Department of the Interior, Bureau of Reclamation; and BC Hydro. He is listed on the Federal Energy Regulatory Commission's (FERC's) lists of approved Independent Consultants and Potential Failure Modes Analysis (PFMA) facilitators. Mr. France has been a prinicipal instructor for three presentations of a four-day Embankment Dam Design Course for the USACE and for seepage and stability analysis courses for the Association of State Dam Safety Officials (ASDSO) and for two presentations of a three-day course on Embankment Dam Seepage Remediation. In 2010, he received the prestigious President's Award from ASDSO for his contributions to dam safety in the United States.

### Project Experience

**Consultant Review Board, Horsetooth Dam, CO, U.S. Bureau of Reclamation:** Member of a three-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for four large embankment dams located near Fort Collins, CO. The principal dam safety issues were seepage-related, including solutioning of limestone and gypsum foundation rock in the left abutment of one of the dams.

**Principal-in-Charge, Carter Lake Dam New Outlet Works, CO, Northern Water:** Principal-in-charge and senior technical reviewer for design and construction phase engineering services for the new outlet works structure at Carter Lake Dam.

**Project Manager, Preliminary design of Chimney Hollow Dam, CO, Boyle Engineering and Northern Water:** Project manager for evaluation of foundation data and dam types and preliminary design of the proposed new Chimney Hollow Dam for the Windy Gap Firming Project.

**Risk Analysis Facilitator, Antero Dam, CO, Denver Water:** Served as an facilitator for a team that completed a potential failure mode analysis and qualitative dam safety risk assessment for Antero Dam, CO. The risk analysis addressed the full range of potential failure modes for an existing embankment dam.

**Risk Analysis Facilitator, Beaver Park Dam, CO, Colorado Department of Parks and Wildlife:** Served as an facilitator for a team that completed a potential failure mode analysis and qualitative dam safety risk assessment for Beaver Park Dam, CO. The risk analysis addressed seepage and internal potential failure modes and potential risk reduction actions for an existing embankment dam.

**Independent Consultant, Williams Fork Dam, CO, Denver Water:** Independent consultant for the FERC-required five-year safety inspection of Williams Fork Dam, CO.

**Independent Consultant, Dillon Dam, CO, Denver Water:** Independent consultant for the FERC-required five-year safety inspection of Dillon Dam, CO.

**Independent Consultant, Rampart Dam, CO, Colorado Springs Utilities:** Independent consultant for two FERC-required five-year safety inspections of Rampart Dam, CO.

**Board of Consultants, Chilhowee Dam, TN, Brookfield Renewable Energy.:** Member of a three-person FERC-mandated Board of Consultants for evaluation of seepage concerns for an existing sloping core rockfill embankment.

**Technical Advisory Panel, Wolf Creek Dam, KY, U.S. Army Corps of Engineers:** Served as chairman of a Technical Advisory Panel reviewing design and construction of major dam safety modifications for Wolf Creek Dam, which is a Dam Safety Action Class (DSAC) 1 facility – the class of highest dam safety concern for the Corps of Engineers. The modifications were completed to address seepage concernms in the karstic foundation of the embankment section of the dam. The implemented solution was a deep, concrete diaphragm seepage barrier wall. Activities also included serving as an estimator for quantitative risk analyses completed during construction and after completion of the project.

**Technical Advisory Panel, Center Hill Dam, TN, U.S. Army Corps of Engineers:** Served as chairman of a Technical Advisory Panel reviewing design and construction of major dam safety modifications for Center Hill Dam, which is a DSAC 1 facility. The modifications were completed to address seepage concerns in the karstic foundation of the embankment section of the dam. The implemented solution was a deep, concrete diaphragm seepage barrier wall.

**Independent Expert Panel, Isabella Dam, CA, U.S. Army Corps of Engineers:** Served on a team that completed an idependent expert panel review of the 65 percent design of dam safety modifications for Isabella Dam, CA, which is a DSAC 1 facility. The modifications are being designed to address seismic stability and spillway capacity concerns for this existing facility.

**Risk Assessment Team, Success Dam, CA, U.S. Army Corps of Engineers:** Served as an estimator on a team that completed a qualitative dam safety risk assessment for Success Dam, CA, which at the time was a DSAC 2 facility. Potential dam safety concerns related to seismic stability, seepage and internal erosion, and spillway capacity.

**Risk Assessment Team and Facilitator, Herbert Hoover Dike, FL, U.S. Army Corps of Engineers:** Served as an estimator on a team that completed a potential failure mode analysis and qualitative dam safety risk assessment for Herbert Hoover Dike, FL, which is a DSAC 1 facility. In a later stage of this four-year long effort, Mr. France served as facilitator for one part of the risk analysis. Potential dam safety concerns for this 150 mile long embankment structure centerd on seepage and internal erosion potential failure modes.

**Technical Advisory Panel, Martis Creek Dam, CA, U.S. Army Corps of Engineers:** Served on a four-member Technical Advisory Panel reviewing dam safety evaluations for Martis Creek Dam, which at the time was a DSAC 1 facility. Issues of concern were seepage and seismic performance.

**Technical Advisory Panel, Success Dam, CA, U.S. Army Corps of Engineers:** Served on a five-member Technical Advisory Panel reviewing design and construction of major dam safety modifications for Success Dam, which is a DSAC 2 facility.

**Consultant Review Board (CRB), Mormon Island Auxiliary Dam and Other Embankment Dams Associated With the Folsom Project, CA, U.S. Bureau of Reclamation:** Member of Consultant Review Boards providing senior technical review of dam safety evaluations, dam modification designs, and construction for one of the embankment dams that impound Folsom Lake, CA. The principal dam safety issues are embankment and foundation seepage and piping, seismic stability concerns and inadequate spillway capacity. Modifications may include a large fuse plug spillway.

**Consultant Review Board, Lauro Dam, CA, U.S. Bureau of Reclamation:** Serving on a three-person CRB providing senior technical review of dam safety evaluations and dam modification designs for an embankment dam in California. The principal dam safety issue is stability and deformation during an earthquake.

**Consultant Review Board, Horsetooth Dam, CO, U.S. Bureau of Reclamation:** Member of a three-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for four large embankment dams located near Fort Collins, CO. The principal dam safety issues were seepage-related, including solutioning of limestone and gypsum foundation rock in the left abutment of one of the dams.

**Consultant Review Board, Keechelus Dam, WA, U.S. Bureau of Reclamation:** Member of a three-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for an embankment dam located near Cle Elum, WA. The principal dam safety issues were embankment and foundation seepage and piping concerns.

**Consultant Review Board, Wasco Dam, OR, U.S. Bureau of Reclamation:** Served as a single reviewer providing senior technical review of dam safety evaluations and dam modification designs for an embankment dam in Oregon. The principal dam safety issues were embankment and foundation seepage and piping concerns.

**Consultant Review Board, Red Willow and Norton Dams, NE, U.S. Bureau of Reclamation:** Served as a single reviewer providing senior technical review of dam safety evaluations and dam modification designs for two embankment dams in Nebraska. The principal dam safety issues were embankment and foundation seepage and piping concerns.

**Consultant Review Board, Clear Lake Dam, CA, U.S. Bureau of Reclamation:** Member of a two-person CRB that provided senior technical review of dam safety evaluations, dam modification designs, and construction for an embankment dam located in northern, CA. The principal dam safety issues were embankment and foundation seepage and piping concerns. The embankment dam was replaced with a new roller compacted concrete dam.

**Advisory Board Member, BC Hydro:** Serving on an Advisory Board for review of BC Hydro's planned dam safety modifications of Strathcona Dam.

**Advisory Board Member, BC Hydro:** Serving on an Advisory Board for review of BC Hydro's planned dam safety modifications of Ruskin and Blind Slough Dams. A major part of modifications at Ruskin Dam were completed to address seepage issues.

**Potential Failure Modes Analysis Facilitator, Baker Project, WA, Puget Sound Energy:** Facilitator for FERC-mandated potential failure modes analysis for the Baker Project, which includes two concrete dams, two powerhouses, and two embankment dams.

**Potential Failure Modes Analysis Facilitator, Noxon Rapids Project, MT, Avista Corporation:** Facilitator for FERC-mandated potential failure modes analysis for the Noxon Rapids project, which includes a concrete gravity intake/powerhouse structure, a gated concrete gravity spillway structure, and two embankment dams.

**Potential Failure Modes Analysis Facilitator, Blue Ridge Dam, AZ, Phelps Dodge Morenci, Inc.:** Facilitator for FERC-mandated potential failure modes analysis for Blue Ridge Dam, a 170-foot high concrete thin-arch dam.

**Potential Failure Modes Analysis Facilitator, Murray Hydroelectric Facility, AR, City of North Little Rock, AR.:** Facilitator for FERC-mandated potential failure modes analysis for a hydro electric facility in Arkansas, located adjacent to a U.S. Army Corps of Engineers lock and dam.

**Potential Failure Modes Analysis Facilitator, Ellis and Whillock Hydroelectric Facilities, AR, Arkansas Electric Cooperative Corporation.:** Facilitator for FERC-mandated potential failure modes analysis for two hydro electric facilities in Arkansas, located adjacent to a U.S. Army Corps of Engineers lock and dam projects.

**Potential Failure Modes Analysis Facilitator, Hydroelectric Station No. 2, AR, Arkansas Electric Cooperative Corporation.:** Facilitator for FERC-mandated potential failure modes analysis for a hydroelectric facility in Arkansas, located adjacent to a U.S. Army Corps of Engineers lock and dam project.

**Toker Dam, Eritrea, East Africa:** Project manager for design and construction of a new, 210-foot-high RCC gravity dam, in Eritrea. The design included preparation of complete plans and specifications for solicitation of tenders from international construction firms. Dam construction was completed in the summer of 1999, at a cost of about $20 million.

**Project Manager, New Construction, Elmer Thomas, USFWS, OK:** Managed field investigations and conceptual and final designs of dam safety actions for an existing 97-foot-high earthfill/rockfill dam. Completed final design of a new 113-foot-high RCC replacement dam.

**Project Manager, Dam Safety Modifications, McKinney Lake Dam, USFWS, NC:** Managed conceptual and final designs of dam safety modifications for an earthfill embankment dam and provided engineering services during construction. The modifications included RCC embankment overtopping protection.

**Project Manager, Dam Safety Modifications, Umbarger Dam, USFWS, TX:** Managed conceptual designs of dam safety modifications for an earthfill embankment dam. The preferred alternative included RCC embankment overtopping protection.

**Project Manager and Facilitator, Workshop on Seepage Through Embankment Dams:** Organized, managed, and facilitated a workshop sponsored by the Federal Emergency Management Agency and the Association of State Dam Safety Officials. The purposes of the workshop were to identify the state-of-the-practice for analysis, evaluation and design related to seepage through embankment dams and to develop prioritized lists of recommended research and development activities to improve the state-of-the-practice.

**Project Manager and Facilitator, Workshop on Dam Outlet Works:** Organized, managed, and facilitated a workshop sponsored by the Federal Emergency Management Agency. The purposes of the workshop were to identify the state-of-the-practice for analysis, evaluation and design related to seepage through embankment dams and to develop prioritized lists of recommended research and development activities to improve the state-of-the-practice.

**Instructor, Potential Failure Modes Workshop:** Instructor for a two-day workshop developed for the Colorado Dam Safety Office.

**Advisor, Risk Prioritization Workshop:** Advisor for a one-day workshop on development of a dam safety risk prioritization tool for the New Mexico Dam Safety Office.

**Instructor, Internal Erosion Potential Failure Modes Workshop:** Instructor for a one-day workshop presented at ASDSO’s National Conference.

**Instructor, Seepage Analysis for Embankment Dams Workshops:** Instructor for two one-day workshops presented at ASDSO's National Conference.

**Instructor, Seepage Analysis for Embankment Dams:** Instructor for a two-day course for ASDSO.

**Instructor, Slope Stability Analysis for Embankment Dams:** Instructor for a three-day course for ASDSO.

**Instructor, Embankment Dam Design:** Instructor for a four-day course for the U.S. Army Corps of Engineers.

**Instructor, Dam Seepage Rehabilitation:** Instructor for a three-day course for the U.S. Army Corps of Engineers.

# Alvi Associates, Inc.

Irfan A. Alvi, P.E.
**President & Chief Engineer**

## RESUME

### Education

B.S. (Honors), 1989, Civil Engineering
University of Maryland

More than 70 credits of post-graduate coursework in engineering, risk analysis, geology, physics, chemistry, biology, and other topics

### Professional Registrations

1994, PE, MD, 20775
1999, PE, VA, 033361
2012, PE, DE, 18065
2015, PE, WV, 021527
2016, PE, PA, 084842

### Relevant Areas of Experience

New Dam Design
Dam Rehabilitation Design
Hydraulic Structures
Dam Inspection
Materials Testing
Forensic Investigation
Human Factors Investigation
Reservoir Routing Analysis
Open Channel Hydraulics
Erosion Analysis
Seepage and Stability Analysis
Structural Engineering
Geotechnical Engineering
Risk Analysis
Dam Removal Study
Fluvial Geomorphology
Construction Management

### Summary of Experience

Mr. Alvi has 28 years of multidisciplinary experience in structural, water resources, geotechnical, and transportation engineering for dams, hydraulic structures, and other infrastructure. In his role as Chief Engineer, he also teaches mechanics of structures, fluids, soil, and rock to all of the engineers at Alvi Associates, at a conceptual depth beyond that offered in typical university courses and textbooks.

Mr. Alvi has completed many hundreds of projects involving inspection, materials testing, forensic investigation, studies, remedial design, and new design. Many of these projects have involved providing innovative solutions to meet challenging situations, with the result that many of his projects have received design awards during the past decade.

Mr. Alvi has nationally-recognized expertise in dam engineering. He served as technical leader for Alvi Associates' Prettyboy Dam project, which received the *2010 National Rehabilitation Project of the Year Award* from the Association of State Dam Safety Officials (ASDSO), which is among the most prestigious awards attainable in the dam engineering profession. The project also received three other awards in 2011, including an ASCE/MD Outstanding Civil Engineering Achievement Project award, ACEC/MD Engineering Excellence Outstanding Project Award, and ESB Outstanding Engineering Achievement Award.

More generally, Mr. Alvi's experience with dam projects has involved diverse technical aspects including inspection, materials testing, forensic investigation, hydrologic and hydraulic analysis, reservoir routing and spillway capacity analysis, dam break modeling and inundation mapping, stream geomorphic study and restoration design, fish passage design, seepage and stability analysis, three-dimensional structural analysis, remedial design, design of new concrete and embankment dams, evaluation and design for dam removal, and construction management.

Mr. Alvi is also nationally-recognized as a pioneer and leader in the role of human factors in dam failure and safety, including organizational and industry aspects. He has served on the ASDSO Dam Failures and Incidents Committee (DFIC) since 2010, leading the committee's work on human factors, making numerous presentations on human factors at ASDSO conferences (including a keynote address) and publishing several peer-reviewed papers. He also served as a Technical Advisor and Human Factors Expert for a FEMA project related to dam failures and incidents. In addition, at the request of ASDSO, in 2015 he presented a two-hour webinar (link) on human factors in dam failure and safety, as part of ASDSO's expert series of webinars. Several of his investigations of dam failures and the associated role of human factors are described below.

### Examples of Dam Study and Design Projects

**Prettyboy Dam in Baltimore, Maryland.** Lead Engineer for aspects related to the gatehouse of this large high-hazard concrete gravity dam which is a key component in the water supply system for the City of Baltimore. Performed review of extensive records related the dam's construction and history (including previous crack monitoring and investigations), abovewater and underwater inspection using an ROV in order to prepare detailed defect mapping, concrete coring and testing, forensic investigation of structural cracking using three-dimensional structural analysis (accounting for creep effects related to structure/foundation interaction) and an innovative causes/effects matrix model, and gatehouse stability analysis considering a wide range of potential failure surfaces.

Based on the findings of this investigation and analysis, performed remedial design for a $6 million post-tensioned anchorage system installed underwater in water depths up to more than 100 feet and consisting of 38 anchors drilled up to 70 feet into the dam (the first system of this type in the world). Also performed contractor prequalification, and extensive construction-phase services including development and evaluation of a preproduction anchor testing program. This 15-year project received four major design awards, as noted above.

**Mill Pond Dam in Cecil County, Maryland.** As a Senior Engineer, participated in alternatives studies and preliminary design for dam reconstruction to address breach in 1999 of an embankment dam dating to circa 1837. Alternatives included elements such as a new twin-cell box culvert outlet structure with a multi-stepped weir and a fish ladder, reconstruction of the failed embankment, embankment widening to allow a wider roadway, roadway reconstruction, a new sheet pile wall, riprap slope protection, and measures to control seepage, piping, and erosion within the new and re-used portions of the embankment dam.

**Seneca Crossing Dam in Montgomery County, Maryland.** Lead Engineer for design for a new concrete gravity dam flanked by embankment dams at each abutment. The concrete gravity dam was selected in order to minimize the dam footprint, and thus reduce the impact to wetlands. Due to adverse subsurface conditions involving highly compressible and permeable materials, an innovative design founding the dam on steel piles was developed and a sheet pile cutoff wall extending 18 feet deep was designed for seepage and uplift control. This design is estimated to have reduced construction costs by at least 40% relative to a conventional concrete dam.

**Bishopville Pond Dam in Bishopville, Maryland.** As Project Manager, to address fish passage needs on a tidal waterway, performed inspection of an existing steel sheet pile dam, tidal hydrologic and hydraulic analysis using TR-20 and HEC-RAS to assess feasibility of dam removal and floodplain impacts (accounting for an existing bridge in the model), and design of a new offline pond isolated via an embankment in order to meet recreational needs of local residents.

### Examples of Dam Forensic and Human Factors Investigations

**Prettyboy Dam in Baltimore, Maryland.** This high-hazard concrete gravity dam is founded on micaceous schist, and is 150 feet high and 700 feet long.

By 1978, extensive cracking was observed in the gatehouse and the adjacent main body of the dam, along with substantial water leakage into the gatehouse stairwell. To respond to this concern, continuing until 1994, six investigations of the cracking had been performed by five previous consultants, but with inconclusive and/or inconsistent findings.

This lead to the forensic investigation for which Mr. Alvi served as Lead Engineer. This involved forensic structural/geotechnical investigation of the gatehouse cracking, eventually discerning that the cracks clustered into eight distinct groups, and likewise discerning three distinct general causes of the cracking, with each cause contributing in varying degrees to each crack group. In other words, a "cause-effect matrix" was developed, thus transcending the usual assumption of a simple one-to-one influence of cause to effect. The three identified causes of the cracking were vertical flexure of the dam, differential settlement between the gatehouse and main body of the dam, and deformation from the reactions of the bridge spans adjacent to the gatehouse. The hypothesized causal matrix was quantitatively validated by analyses of stresses and deformations of the dam, gatehouse, and bedrock, and the resulting predictions were found to fit the observed cracking remarkably well.

More broadly, preceding the design phase, the project involved a comprehensive multi-phase dam investigation involving many tasks: exhaustive review and summary of all available records, abovewater inspection, underwater inspection using divers and a remote-operated vehicle (ROV), precise mapping of defects throughout the exterior of the dam as well as inside the gatehouse, crack monitoring during gate testing operations, concrete coring and testing, analyses and evaluations, and preparation of a 300-page study report with recommendations.

The findings of the investigation were presented in the report for the client, a peer-reviewed paper in the ASDSO *Journal of Dam Safety* (link), and a presentation at the ASDSO national conference.

**Big Bay Dam in Mississippi.** This embankment dam was over 50 feet high and 2000 feet long, and failed in 2004, resulting in damage or destruction of more 100 structures. As Lead Human Factors Investigator, performed a comprehensive investigation of the failure, including review of many hundreds of pages of documents, including plans, calculations, construction records, deposition transcripts, engineering reports, etc. Focusing on the human factors aspect of the failure, identified the roles of the engineer, owner, state regulatory agency, maintenance personnel, and inspectors, as well as the complex interaction of human factors and physical factors during the two decades from the design until the failure. Findings of the investigation were presented in a peer-reviewed paper in the ASDSO *Journal of Dam Safety*, a dedicated 'soapbox' session at the ASDSO national conference, an ASCE invited speaker presentation (link), and Mr. Alvi's 2015 webinar for ASDSO.

**Sella Zerbino Secondary Dam in Italy.** The Sella Zerbino secondary dam was a concrete gravity dam about 46 feet high and 360 feet long. In 1935, a decade after construction, the dam failed catastrophically, resulting in at least 111 fatalities. Starting with the planning of the project four decades prior to the failure, a series of human and physical factors interacted and compounded, until a 1000-year storm was the final physical trigger for the failure. Additional physical factors included lack of a spillway for the secondary dam, instability and erodibility of the foundation rock at the secondary dam, and grossly inadequate discharge capacity for the reservoir, which was exacerbated by clogging of spillways and outlets. The human factors contributing to the failure included hasty design and construction of the secondary dam after a late decision to raise the height of the main dam, inadequate geologic investigation and missed warning signs related to the foundation of the secondary dam, and lack of rainfall data to adequately design spillways and outlets. Focusing on human factors, this investigation involved an extensive literature review, mapped out the role of physical factors, and contributed new insights into the failure by identifying the role of human factors in the failure, using the framework pioneered by Mr. Alvi. Findings of this investigation were presented in a peer-reviewed 2015 paper (link) and presented at an ASDSO national conference.

**St. Francis Dam in California.** This arched concrete gravity dam near Los Angeles was nearly 200 feet high, and failed in 1928, about four years after construction began and a day after fully filling the reservoir for the first time, resulting in a flood which extended more than 50 miles and resulted in at least 400 fatalities, along with millions of dollars of property damage. The failure is considered by many to be the worst US civil engineering disaster of the 20th century. Focusing on human factors, performed a comprehensive investigation of the failure, including review of hundreds of pages of documents, including plans, engineering analyses, other investigations, etc. Focused on the human factors aspect of the failure, identifying the roles of the chief engineer, other engineers working under the chief engineer, City of Los Angeles, and local citizens who reported warning signs, as well as the complex interaction of human factors and physical factors during the years preceding the failure. Findings of the investigation were presented in a peer-reviewed 2013 paper (link) and a presentation at the ASDSO national conference.

**Ka Loko Dam in Hawaii.** This embankment dam was 42 feet high and 770 feet long, and failed in 2006, resulting in flood depths of 10 to 30 feet, seven fatalities, extensive property and environmental damage, a criminal sentence for the owner, and a civil settlement of many millions of dollars. Performed a comprehensive investigation of the failure, including extensive literature review of many hundreds of pages of documents, including plans, calculations, engineering reports, other investigations, news reports, etc. Focused on the human factors aspect of the failure, identifying the roles of the owner, Corps of Engineers, a trust which owned a portion of the reservoir, the County and Mayor, state regulatory agency, federal regulatory agencies, maintenance personnel, and inspectors, as well as the complex interaction of human factors and physical factors during the century preceding the failure. Findings of the investigation were presented at an ASDSO national conference, a keynote address at an ASDSO conference (link), and Mr. Alvi's 2015 webinar for ASDSO.

# PETER A. DICKSON

**POSITION IN FIRM:** Vice President, Global Geotechnical Practice Leader, Geotechnical Risk Manager

**YEARS WITH FIRM:** 39 **TOTAL YEARS OF EXPERIENCE:** 44

**KEY QUALIFICATIONS:**

Dr. Peter Dickson has broad experience on a large variety of water resource projects in many parts of the world mostly involving dams for water supply and hydroelectric projects, spillways, power plants, penstocks, tunnels, caverns, and flood control structures. His 39 years with MWH has involved a particular emphasis on large hydropower and pumped storage projects. His experience includes project screening and ranking; design and supervision of geological and geotechnical investigations; siting of project features and developing layouts and arrangements; dam type selection, slope stability evaluations and slope design; foundation design for heavy civil works; criteria for planning, design and construction of dams, spillways, and underground works; detailed design and contract document preparation; hazard and risk analysis, including forensic investigations.

As Global Geotechnical Leader, his work involves coordination of work being done by geotechnical teams in offices around the globe, including implementation of consistent practices, and establishing consistent communications between offices. His responsibilities include assessment of technical risk on new pursuits, risk assessment on various projects during implementation using PFMA and FMEA methods, independent technical review, and constructability review.

**EDUCATION:**

PhD, Geology, University of Pittsburgh, 1977
MSc, Geology, University of Manchester (UK), 1974
BSc, Geology, University of Leeds (UK), 1971

**Professional Registration:**
Professional Geologist, Georgia, 1990
Professional Geologist, Virginia, 1984
Professional Geologist, Wyoming, 1993
Professional Geologist, Indiana, 1983

**SELECTED DAM AND HYDROLECTRIC PROJECT EXPERIENCE:**

**Neelum-Jhelum Hydroelectric Project, Pakistan**
*Client: WAPDA*
Currently serves as member of senior technical review team during detailed design and construction of 979-MW hydroelectric project scheme in Himalaya foothills, Pakistan. Responsible for site visits, value engineering and technical advice to design and construction teams, technical review of underground works (>56km tunnels, underground powerstation complex, 300-m-high surge shaft), and concrete gravity dam, de-sanding chambers; tunnel construction by drill-blast and TBM. Severe project challenges include major earthquake hazard, weak foundation materials, active fault crossings for tunnels, highly deformed and complex tunnel geology. Technical lead in evaluation of earthquake hazard and foundation design; development of seismic design parameters for final design including ground motion estimates, response spectra and time histories.

**Priest Rapids Dam, Washington**
*Client: Grant County Public Utility District*
Independent Technical Reviewer of investigations and detailed design of remedial works under right side of embankment dam, where existing dam is founded on potentially liquefiable materials. Provided guidance to client and design team in selection of a preferred solution involving construction of a new dam downstream of the existing dam with a core tie-in by construction of a cutoff wall; presented findings and design solution to FERC.

**Wanapum Dam Spillway Remediation Project, Columbia River, Washington**
*Client: Grant County Public Utility District*
Independent Technical Reviewer for repairs and stability improvements to the Wanapum Dam Spillway following discovery of serous vertical and downstream movement in one of the 13 spillway monoliths. Provided independent review and oversight of forensic study of the root cause of the movements, foundation analysis, finite element static and seismic stress analyses, and stability improvements to the entire spillway. Final solution included installation of 61 strand tendon post-tensioned anchors and 3” dia post-tensioned bar anchors in each spillway monolith.

**Panama Canal 3rd Set of Locks Project, Panama**
*Client: GRUPO and Autoridad del Canal de Panama (ACP)*
Dr. Dickson served on technical review board for the designers of the design-build team for construction of new locks expansion project. Responsible for review of all foundation and seismic aspects, geotechnical design criteria, design procedures and analytical approaches, risk assessment, and submittals. Following successful bid, continued in this role during construction in review of foundations and seismic aspects, including forensic examination of root cause of foundation problems occurring during construction. At Client request, and prior to bidding period, served as peer reviewer of seismic hazard and geologic data acquisition and interpretation, and advised client in approach to selection of seismic design criteria. During design and construction, reviewed foundations, RFIs, changed conditions issues, and submittals. Independent reviewer during preparation of design and drawings for repair of cracked lockhead structures; was involved in root cause analysis.

**Tekeze Dam, Ethiopia**
*Client: Ethiopian Electric Power Corporation*
Responsible for field inspection of project works and seismic hazard on 190-high thin-arch concrete dam under construction, responsible for identification of foundation and rock slope stability critical issues, rock mechanics evaluations (including 3-D rigid block and kinematic analyses, block theory, 2-D and 3-D FEM stress and deformation analysis). Responsible for rock reinforcement and anchor design, including thrust-block design and left abutment remedial works (abutment replacement), review of underground works (power tunnel system and powerhouse cavern); QA of site engineering.

**Iowa Hill Pumped-Storage Project, California**
*Client: SMUD (Sacramento Municipal Utilities District)*
Lead geological engineer in re-examination of project layouts and construction costs. Prepared new revised layouts for underground works including powerstation complex, high pressure and low pressure waterways, power shaft, and upper reservoir. Prepared risk assessment and conducted FMEA workshop. Participated in preparation of estimates of construction costs, constructability analysis, and scheduling. Advised on development of scope of additional site investigations. (Intermittent 1990 to 2013)

**John Hart Hydroelectric Project and Ruskin Hydroelectric Project, British Columbia, Canada**
*Client: BC Hydro*
Served on team providing a validation of replacement of the existing John Hart Project and Ruskin Project, both of which exhibited serious technical deficiencies. Role included evaluation of geological conditions at sites, foundation geotechnical assessment, potential seepage and piping issues, seismic hazard evaluation, and review of proposed engineering solutions to deficiencies. With client staff, conducted detailed risk assessment and facilitated failure modes workshops (FMEA).

**Shandong Taian Pumped-Storage Project, China**
*Client: Shandong Taian Pumped-Storage Power Station Co. Ltd*
Consultant responsible for checking review and technical advice on design and construction of upper reservoir CFRD dam and underground works. Responsible for assessment of site investigations, check of design criteria, tunnel design, independent analysis and assessment of stability of caverns and tunnels, and shafts; conducted numerical analysis, excavation and support design, overall review of design and construction methods; review and advise construction design of underground works, including review construction procedures and specifications; check of project construction schedule, implementation plan, and construction cost estimates; responsible for technical advice and evaluation of geomembrane lining system for upper reservoir. (2001 – 2007)

**Seneca Station Pumped-Storage Project, Pennsylvania**
*Client: First Energy*
Served on Risk Evaluation Team during facilitated Probable Failure Modes Analysis (PFMA) workshop for hydroelectric pumped storage project. The purpose was to evaluate risks associated with piping failure, slope stability, structural concerns, and operations of the project. A total of 27 potential failure modes were evaluated, including risk reduction measures; prepared report. Lead geological engineer and quality control engineer during emergency and forensic investigations, development of repair program, drawings and specifications, and implementation of repairs at lined upper reservoir of pumped storage project. Developed program to repair cracked asphaltic-concrete reservoir floor lining. Responsible for supervision and review of design components, Health and Safety Plan, QCIP, and field supervision.

**International Panel of Experts, Iraq**
*Client: Ministry of Water and Energy Resources*
Expert in geological engineering, karst, and seismicity serving on International Panel of Experts (POE) for Minister of Water and US Embassy, to provide technical review, advice, and oversight of all dam projects in operation, under design, or investigation for planning and future development. For three years the main focus of the panel was on the management and safety of Mosul Dam, including subsurface and hydrogeological investigations, evaluation of alternatives, review of on-going grouting programs and instrumentation/monitoring. The POE provided a long-term solution to the Owners of this extreme risk project which they have now accepted and is in the process of being implemented.

**San Pedro Hydroelectric Project, Chile**
*Client: Colbun*
Dr. Dickson served on Board of Consultants as technical expert on foundations and underground structures. Assignments involved review of a $250-million 144-MW hydroelectric project in Chile with installed capacity of 144 MW. Review activities included site visits and detailed inspection of investigations and design with Owner and its design team. The project is in early stages of construction and opinions were provided regarding constructability, schedule, technical and construction risks.

**Proyectos HidroAysen, Chile**
*Client: HidroAysen*
Served on Board of Consultants as technical expert on foundations and underground structures. Assignment involved review of five hydroelectric projects under various stages of development with total installed capacity of 2750 MW. Review activities included site visits and detailed inspection of investigations and design review with Owner and its design team. Helped in risk workshops using risk-informed decision-making processes (including development of event trees and fragility curves in risk-based models) to assist in determining design criteria for critical project components. Construction: On hold pending environmental studies financing. Project Cost: $3.5B

**Sanchung Pumped-Storage Project, Korea**
*Client: Hyundai Engineering Company, Ltd., Seoul, and Saman Engineering Co.*
Consulting geologist during construction of upper and lower reservoir dams and underground works. Responsible for evaluation of rock support system, geologic mapping and detailed geologic data collection in newly excavated works. Carried out stability and finite element analyses of stresses around PH cavern. Results used to check rock support provisions and recommend modifications in support system, underground construction methods, grouting and drainage. During planning and design phase, independent checking engineer responsible for assessment of site investigations, design oversight of all geotechnical features including power tunnel system, PH, and surge shaft, two concrete-faced rock fill dams, spillways, and material quarries; carried out numerical analysis of cavern and tunnels, recommended underground layout revisions, additional investigations, design details, prepared rock support system, and underground construction methods; dam foundation assessment, recommended additional investigations and grouting requirements.

**Al Wehdah Dam, Jordan**
*Client: Jordan Valley Authority*
Lead engineering geologist for the $400-million water supply dam project. Major structures include a 140-m-high, concrete-faced rockfill dam (CFRD), which was later re-designed as a roller-compacted concrete dam (RCC); 400-m-long chute spillway; penstocks; irrigation tunnels; and power plant. The dam site has sensitive foundations with karstic

limestone, highly soluble altered rock, weak marls and clay layers. During final design and construction bid document preparation, responsible for site investigations, selection of geologic design parameters including foundation stability and treatment, evaluation of spillway foundation potential failure modes, earthquake hazard evaluation, and input to cost estimate. During construction, evaluated foundations, conducted stability assessments (including 3-D block analyses), rock mechanics analyses, proposed design and construction modifications, reviewed foundation treatment and grouting works. Resident engineer during construction of diversion tunnel; provided construction supervision for treatment of karst features in foundations; evaluated reservoir leakage potential and reservoir rim stability.

**El Cajon Hydroelectric Project, Mexico**
*Client: CFE and CIISA*
Principal engineering geologist serving on Dispute Resolution Board retained by Owner (CFE) and contractor consortium (CIISA) to provide expert opinion and resolve financial disputes upon completion of construction of major hydroelectric project with high concrete-faced dam, underground powerstation complex. Activities included site visit, meetings with Owner and Contractor, review of technical and contract information, and development of opinion on validity of claims involving changed/unanticipated geologic conditions, scheduling interferences, cost impacts (2007)

**Karahnjukar Hydroelectric Project, Iceland**
*Client: Landsvjirkjun*
Lead geological engineer in review and finalization of seismic design for 200-m-high concrete-faced-rockfill dam in Iceland; QA review of geology, foundations, and grouting. Presented findings and recommendations to independent technical review Panel of Experts.

**Diamer Basha Dam and Hydroelectric Project, Pakistan**
*Client: WAPDA*
Project involves a 270-m-high dam on the Indus River in highly seismic region of the Himalayas. Responsible for seismic hazard evaluation and preparation of seismic design parameters and criteria. Developed initial layouts and basic design of underground works, including two 6-unit powerhouses, transformer/GIS caverns, surge chambers, power tunnels, tailrace tunnels, and construction access tunnels. Provided technical review and input in geotechnical investigations, foundation design, and detail design requirements of underground structures. Responsible for detailed geotechnical and rock mechanics analyses including finite element modeling of caverns, rock support design excavation design, stability assessment of spillway plunge pool, tunnel lining design parameters, rock slope analysis and design, supervision of geotechnical design team, preparation and review of reports. Task leader in independent study of optimizing project arrangements, dam type selection and criteria.

**Boyabat Dam Project, Turkey**
*Client: Doğuş Construction and Trading Co.*
Lead geotechnical engineer during final design of project involving a 200-m high concrete gravity dam on Kizilirmak River in north-central Turkey. The 3-unit project has an installed capacity of 528 MW. Significant issues related to presence of faulted, karstic limestone in the foundation and abutments, and due to the proximity of the North Anatolian Fault, capable of M = 8.0 events. Responsible for site visits, directing a team of geologists and geotechnical engineers in final design including foundation evaluation and stability analysis, foundation excavation and treatment design, cofferdam design, instrumentation design, earthquake analysis, specifications and drawing preparation.

**Bath County Pumped-Storage Project, Virginia**
*Client: Virginia Electric Power Company*
Field geologist responsible for foundation investigations (lower reservoir dam and spillway, upper dam), stability analysis of lower reservoir spillway foundation, detailed geologic mapping of excavations and power tunnels and site inspection during construction; member of grouting team during emergency repairs to power tunnel system, included review of geologic and grouting data in high-pressure tunnel and penstock areas; interpreted Thermal Infrared Imagery to enable detection and monitoring of leakage and springs.

**CURRICULUM VITAE RELATIVE TO SPILLWAYS**

**Name:** Henry T. Falvey

**Nationality:** US citizen

**Profession:** Consultant in Hydraulic Engineering

**Key Qualifications:**

- Wrote a book on labyrinth weirs that was published by ASCE.
- Consultant for Freese Nichols for the design of the labyrinth weir on the Brazos River at Waco TX. I proposed modifications to the design, participated in physical model study and inspected installed labyrinth.
- Analyzed the flow capacity for the Flamingo Dam labyrinth spillway at Las Vegas, NV.
- Analyzed labyrinth design methods to increase spillway capacity for Los Angeles District Corps of Engineers at Prado Dam. Included site visit to evaluate existing design, development of design parameters, and advised on model studies at the Waterways Experiment station.
- Independent Technical Reviewer for Sacramento District Corps of Engineers of Isabella Dam labyrinth spillway. Included site visit and observation of physical model study at the Water Research Laboratory of Utah State University.
- Prepared an engineering monograph for the analysis of cavitation problems of chutes and spillways for Bureau of Reclamation. The monograph includes recommendations for repair, surface tolerances, aeration, and several computer programs to analyze cavitation potential and aerator design.
- Evaluated cavitation performance and recommended remedial measures on Blue Mesa, Flaming Gorge, Glen Canyon, Hoover, Kortes, and Yellowtail Dams for Bureau of Reclamation and Seven Oaks Dam for the Corps of Engineers.
- Hydraulic consultant to Hydroplus on the use of fusegates to increase spillway and reservoir capacity.
- Analyzed cavitation potential for Manenggon Hills Dam spillway for Dames & Moore.
- Analyzed spillways of Tolt, Railroad, and Smith Lake Dams, as well as the outlet works of Twin Lakes Dam for Woodward Clyde. These included investigations of existing spillways and spillways proposed to increase capacity.
- Designed baffled apron drop to control saturation at Ralston Reservoir and analyzed Dillon Dam morning glory spillway for Denver Water Board.
- Independent Technical Reviewer for US Corps of Engineers on outlet works for Folsom, Seven Oaks and Coyote Dams, spillway of Kaweah Dam and auxiliary spillway for Folsom Dam, bypass structures for Guadalupe River in San Jose, field-testing of Warm Springs Dam, fish bypass for the Dalles Dam, and spillway for Folsom Dam.

- Independent Technical Reviewer for URS Australia on Burnett (now Paradise), Eildon, Dartmouth and Hinze Dam spillways.
- Consultant for HKM Consultants in Billings Montana on the spillway and outlet works upgrade for Bair, Ruby and Nevada Creek Dams.
- Analyzed allowable surface tolerances on spillway surface for Granite Construction Company at Prado Dam.
- Directed physical model study of the double curvature arch dam Eagle Nest Dam for overtopping in NM for URS.
- Analyzed erosive damage potential for spillway flow from Warragamba Dam in Australia.
- Member of review board to reduce total dissolved gas content from the spillways on Boundary Dam in the state of Washington and on Cabinet Gorge Dam in Idaho.
- Author of the following papers on hydraulics of spillways and outlet works.

**Engineering monographs**

"Air-Water Flow in Hydraulic Structures," US Bureau of Reclamation Monograph No. 41, 143 pp., 1980.

"Cavitation in Chutes and Spillways," US Bureau of Reclamation Monograph No. 42, 143 pp., 1990.

**Papers**

"Hydrodynamic Pressures in Conduits Downstream of Regulating Gates," *IAHR Congress*, Fort Collins, CO 1976.

"Mean Air Concentration of Self-Aerated Flows," ASCE *Journal of the Hydraulics Division*, Technical Note, Vol. 105, No. HY1, pp. 91-96, 1979.

"Predicting Cavitation in Tunnel Spillways," *International Water Power and Dam Construction*, pp. 13-15, Aug. 1982.

"Prevention of Cavitation on Chutes and Spillways," ASCE *Proceedings of the Conference on Frontiers in Hydraulic Engineering*, Cambridge, MA, pp. 432-437, Aug. 1983.

"Tests on Cavitation Inception from Cylindrical Holes in a Boundary," (Co-authored with B. Mefford), ASCE *Hydraulics Division Specialty Conference*, Coeur d'Alene, ID, Aug. 1984.

"Increasing Shongweni Dam Discharge Capacity with a Hydroplus Fusegate System," (Co-authored with E.F.A. Snell and J. Raussiguier), *Proceedings of the USCOLD Annual Meeting*, Phoenix, AZ, 1994.

"Hydraulics and Design of Fusegates," (Co-authored with Treille), ASCE *Journal of Hydraulic Engineering*, Vol. 121, No. 7, 1995.

"Case Study - Dillon Dam Trashrack Damage," (Coauthored with J.H. Weldon), 2002, *ASCE Journal of Hydraulic Engineering*, Vol. 128, No. 2. , pp. 144-150.

“Investigation and Repair of the Outlet Works Tunnel Slab Damage - Seven Oaks Dam, Santa Ana River, California,” *Poceedings of the IAHR International Symposium on Hydraulic Structures*, Ciudad Guayana, Venezuela, Oct. 2006 ( Co-authored with Burgi, P.H., Cozakas, D.P., Kwan, R., Sawka, M.J., Schlenker, S.J. and Waller, T.N.)

**Book**

*Hydraulic Design of Labyrinth Spillways*, ASCE Press, 2002.

**Membership**

Member of International Association for Hydraulic Research (IAHR),
American Society of Civil Engineers (ASCE), United States Committee on Large Dams (USCOLD
Associate editor for Journal of Hydraulic Engineering ASCE.
Member of Hydraulics Committee of USCOLD and Unsteady Flow in Hydraulic Machinery Work Group of IAHR..
Consulted or taught in Algeria, Australia, China, Egypt, France, Germany, India, Mexico, Pakistan, Romania, Switzerland, Taiwan, and Turkey.

**Education:**

| | |
|---|---|
| BSCE with Honor | Georgia Institute of Technology |
| 1953-1958 | Atlanta, Georgia |
| MSCE | California Institute of Technology |
| 1959-1960 | Pasadena, California |
| Dr.-Ing. | Universität Karlsruhe |
| 1962-1964 | Karlsruhe, Germany |

**Experience Record:**

1960-1962: Hydraulic Engineer, Division of Research, US Bureau of Reclamation, Denver, CO
1962-1964: Student, Universität Karlsruhe Germany
1964-1970: Hydraulic Engineer, Division of Research, US Bureau of Reclamation, Denver, CO
1970-1972: Head, Hydraulic Research Section, Division of Research, US Bureau of Reclamation, Denver, CO
1972-1974: Senior Research Officer, Ecole Polytechnique Federale de Lausanne, Switzerland.
1974-1987: Technical Specialist, Division of Research, US Bureau of Reclamation
1987-1991: Private Consultant and Faculty Affiliate, Colorado State University
1991-present: President, Henry T. Falvey & Associates, Inc. and Faculty Affiliate, Colorado State University.

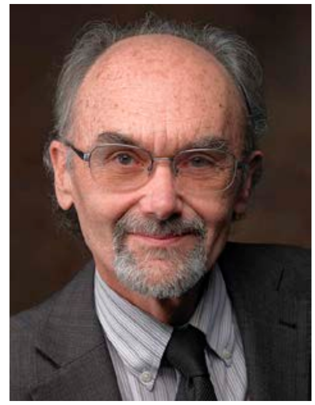
**Languages:**

| | Speaking | Reading | Writing |
|---|---|---|---|
| English | Native | Native | Native |
| German | Excellent | Excellent | Good |
| French | Fair | Good | Poor |

**Honors:**

- Bureau of Reclamation - Silver Medal for Meritorious Service.
- Nominated for the Denver Federal Executive Board Outstanding Scientist/Engineer Award.
- Denver Federal Center Professional Engineers Group - Engineering Achievement Award.
- American Society of Civil Engineers - Hydraulic Structures Medal.
- American Society of Civil Engineers - Best paper in Division of Irrigation and Drainage, 1993.
- Society for Technical Communication - Distinguished Technical Communication Award for *Air-Water Flow in Hydraulic Structures* (1981) and *Cavitation in Chutes and Spillways* (1991).
- International Television Association - Award of Excellence for Video *Cavitation - A Bursting Bubble*.
- College Awards - Chi Epsilon, Tau Beta Pi, Briarian Society, Distinguished Military Graduate.
- Holder of one patent and applied for another.
- Recognized as one of eleven eminent water resources engineers who have made a great contribution to the profession during their career at the 2004 American Academy of Water Resources Engineers meeting in Anchorage Alaska. Bestowed the Academy's highest honor of "Honorary Diplomate."

# Curriculum Vitae - Stephen James Rigbey



## POSITION

Director, Dam Safety, BC Hydro, British Columbia, Canada

President, SJR Consulting Inc., Vancouver, Canada

## EDUCATION

B.A.Sc., and M.A.SC., Geological Engineering,

University of Windsor, Canada, 1975, 1980

## PROFESSIONAL ASSOCIATIONS

Association of Professional Engineers and Geoscientists of the Province of British Columbia

Professional Engineers of Ontario

Canadian Dam Association (CDA) – member Dam Safety Committee

International Congress on Large Dams (ICOLD) – Canadian representative on the Seismic Committee

## SUMMARY OF EXPERIENCE

Stephen joined Acres International (now Hatch) in 1979, where he rose to Principal Geotechnical Engineer and Project Manager. Through Acres, he gained extensive worldwide experience, working in more than 15 countries and following numerous projects through investigations, design, construction of underground complexes, concrete and earthfill dams, and long-term monitoring. Specialties include instrumentation and dam safety, seismicity assessments, and rock mechanics designs.

Stephen then joined BC Hydro in 2008, and is now the Director of Dam Safety, responsible for ensuring safe reservoir retention and passage through and around hydro facilities at 41 sites throughout the province of British Columbia, Canada. He is responsible for monitoring and surveillance, the identification and prioritization of all associated risks, initiating Investigation and Capital projects, and providing technical oversight to these projects, which will total $CAN 1.9B over the current 10-yr Capital Plan. He has a staff of about 35 professionals, technologists and support staff.

Externally from BC Hydro, Stephen has been a member of Advisory Boards for the Lower Churchill Project in Newfoundland and for audits of dam safety programs for utilities in Sweden and Turkey. As a member of the CDA Dam Safety committee, Stephen was heavily involved with the 2013 review and updating of the Dam Safety Guidelines.

Stephen has been recently awarded the prestigious Inge Anderson Award by the Canadian Dam Association in recognition of his “significant contributions to the advancement of knowledge and practices related to dams in Canada”.

- **Operational Safety**: At BC Hydro, Stephen has initiated changes to spillway and reservoir operations in a number of cases as interim risk management measures. He has also initiated overall operational reviews of reservoirs and river systems in view of dam safety considerations. Current work involves the Campbell River System on Vancouver Island, where operations and input hydrographs have been stochastically modelled to better understand the consequences of a single random spillway gate failure to operate on demand. Results have indicated that changes to operational rules will greatly reduce overall downstream flooding risks while only marginally increasing upstream risks. Stephen is also one of the four sponsors responsible for initiating the work behind the recently published book *Operational Safety of Dams and Reservoirs* (Sept 2016, ICE Publishing).
- **Dam Safety Assessments**: Prior to joining BC Hydro, Stephen performed numerous dam safety assessments of various earth and concrete structures within Canada, including detailed foundation condition investigations, coordination of laboratory work, and assessment of rock/concrete shear strengths. A systematic approach to the evaluation of parameters for stability analyses was developed for this work, and is quoted in the Canadian Dam Association Guidelines. Internationally, Stephen has performed Dam Safety Reviews and seismic hazard assessments for high-risk dams in Iran, El Salvador and Panama.
- **Administration and Quality Control** : At BC Hydro, Stephen is responsible for initiating and reviewing all major projects involving water passages and dams. At Acres, he was responsible for the administration and technical coordination of all geotechnical work on hydro projects, including seismicity assessments. Stephen also has significant experience in quality control reviews, and has acted as Lender's Engineer in the review of designs and construction of dams in Panama and India.
- **Dam Investigations and Design** : Stephen has been the Project Manager for the design of a 35-m high, 500-m long embankment dam founded on sands and gravels, including a 65-m deep plastic concrete cutoff wall. He had previously managed an investigations and monitoring program for the original dam, which included an extensive remote automated instrumentation system, development of a detailed Emergency Preparedness and Response Plans, infilling of sinkholes, geophysical surveys, exploratory boreholes through the damaged core of the dam, and sonar surveys of the headpond. A methodology for precise sonar surveying of the sinkholes was developed under Stephen's supervision as part of the work for this project. Stephen has worked in karst environments for projects in both Iran and Indonesia.
- **Rock Mechanics and Geological Engineering** : Stephen has designed layouts and support for underground powerhouses and tunnels under various conditions, including extremely high horizontal in situ stresses and time dependent deformations. Design studies have included the development of specialty laboratory tests to investigate swelling rock pressures. He also has experience in the laboratory and field identification of alkali-aggregate reactions. Stephen spent 2 years on-site in India during the construction of a major underground power facility, and was also Resident Site Engineer during the construction of a 600-m long, 3.5-m dia adit and a 13.5 m dia trial excavation chamber in shales. He was involved onsite at the Karun III project in Iran during the construction of a 200-m concrete arch dam and underground powerhouse complex. He also worked for a number of years on the Niagara Diversion Tunnel project, a10-km long, 14.4-m diameter rock TBM drive, including investigations, design, and the development of the project Geotechnical Baseline Report.

**CAREER CHRONOLOGY**

***SJR Consulting Inc – Vancouver, 2013 to present***

- Advisory Board member, Muskrat Fall Hydroelectric Project, Newfoundland (Nalcor) : project involves RCC and embankment dams about 20 to 40 m high, and a unique abutment issue where a currently meta-stable landform must be transformed into a robust reservoir retaining structure.
- Audits of Dam Safety Programs: extensive audit of management and technical processes and procedures against ICOLD Bulletin B154 - Dam Safety Management: Operational Phase of the Dam Life Cycle, for Vattenfall Vattencraft (Sweden) and EnerjiSA (Turkey), resulting in 40 to 50 specific recommendations for consideration in each case.

***BC Hydro – Vancouver : Manager (now Director) of Dam Safety, 2008 to present***

- Responsible for all aspects of safety involved with water retention and water passage structures at 41 separate hydroelectric sites, developing the appropriate scope and initiating all major studies, investigations and civil projects associated with dam safety, and for providing technical guidance throughout the execution of these projects.
- Reporting directly to the Deputy CEO, and to the Board of Directors on a quarterly basis

***Hatch Energy – Vancouver : Principal Geotechnical Engineer & Project Manager, 2006 to 2008***

- Comprehensive review of shear strength parameters for the Ruskin Dam, including investigations for basic resistance of the bedrock-concrete contact.
- Forrest Kerr Hydroelectric Project Feasibility Study —Responsible for geotechnical investigations and feasibility level designs for a 190 MW underground powerhouse scheme and 3.4 km tunnel in Northern British Columbia.
- Risk Study for Kemano Tunnel, including investigation of submersible inspection techniques. This partially-lined rock tunnel has a history of rock collapses, and is critical to the supply of power for Alcan's aluminum smelter at Kitimat. Various alternatives to reduce failure risk were compared on a Net Present Value Basis.
- Project Manager for various detailed sonar bathymetry studies for BC Hydro and TransAlta.
- Dam Safety Review Engineer for various large dams, including BC Hydro's extreme consequence category Mica Dam, a 240-m high earthfill structure, and the 120 m high Wood Creek Suncor tailings dams.
- External consultant to BC Hydro for an audit of monitoring and surveillance practices within the Dam Safety Group.

***Acres International (later Hatch Energy),*** **Niagara Falls: Principal Geotechnical Engineer**, ***2001 to 2005***

Responsible for quality control of geotechnical work and mentoring/guidance of geotechnical staff within the hydro division. Continued in previous role as Department Head.

Development of the Geotechnical Baseline Report (GBR) for the Niagara Diversion tunnel project (14.4-m excavated dia., 10 km long). Participated in technical evaluation of the bids and contract negotiations.

Seismic hazard assessments and dam safety reviews for high-risk dams, including regional seismicity reviews, deterministic and probabilistic hazard analyses, selection of design events for:

- Dez dam, Iran
- four dams in El Salvador
- Fortuna dam, Panama.

Due diligence studies and site visits as Lender's Engineer for hydroelectric projects during construction:

- Estí project in Panama, a 120-MW scheme involving an earth dam, 6-km canal, 50-m high concrete-face rockfill dam, 4.8-km, 7-m dia tunnel and surface powerhouse
- Vishnuprayag project in India, involving 11-km, 4-m dia tunnel and 400-MW underground powerhouse.

Onsite review of abutment stability safety for a 140-m high, concrete gravity arch dam at the Chamera project, northern India, following a massive downstream landslide.

Project Manager for the design of the 35-m high, 500-m long Shikwamwka Replacement embankment dam, founded on sands and gravels, and incorporating a 65-m deep plastic concrete cutoff wall. Also acted as Project Manager for the monitoring and investigations program involving the original dam.

Numerous dam safety assessments of various earth and concrete structures in Ontario and New Brunswick, including detailed foundation condition investigations, coordination of laboratory work and assessment of rock/concrete shear strengths for the stability analyses. A systematic approach to the evaluation of parameters for stability analyses was developed for this work.

***Acres International, Niagara Falls, Geotechnical Department Head - 1996—2001***

Responsible for the administration and technical coordination of all geotechnical work on all projects involving geology, soil and rock mechanics, and seismicity. The department had a staff of 12-15 engineers and technologists.

Acting Geotechnical Site Engineer, Karun III hydroelectric project, Iran. Responsibilities included

- supervision of foundation preparation in a faulted area for a 205-m concrete arch dam
- providing advice on major ground movements experienced during excavation of the 26-m span underground powerhouse; review of all instrumentation results
- development of 3D numerical rock mechanics models for the underground complex
- development of 2D seepage models for the arch dam abutment

Project Manager responsible for a study to examine possible causes of, and to develop alternative remedial options for, significant leakage at the Old Mill Station subway tunnel portal, Toronto, Ontario. The study included geotechnical investigations, conditional surveys and a full-scale pump test.

Inspection and interpretation of monitoring data, detailed hydrogeological site assessment, and detailed analysis of groundwater chemistry for the Irving Paper aeration and stabilization basin. The basin containment dike is constructed on a soft marine clay deposit, Saint John, New Brunswick. Preparation of yearly monitoring reports are issued for the New Brunswick Ministry of the Environment.

Project Manager for potential rockfall studies on 48 highway rockcuts in eastern Ontario.  Report included recommendations for remediation, cost estimates and benefits.

On-site consultation for remediation of a collapsed 3.5-m dia tunnel in overburden in Bolivia.

Design of remedial works for a 100+ m slope undergoing long-term creep in weathered rock over a water tunnel intake for the Cañon del Pato project in Peru.  On-site consultations and construction reviews.

Provided on-site technical advice for the Second Power Reconstruction project, Bosnia and Herzegovina, in the planning for re-instrumentation of the Bocac and Trebinje dams following damage during the war.  Developed specifications for international tendering through the World Bank.  Instrumentation included precise survey equipment for geodetic monitoring, regional seismicity monitoring and local accelerometer networks, automatic weather stations, and various geotechnical instrumentation systems, including telependula, vibrating wire piezometers, strain gauges, tiltmeters, and ADAS systems.

Vibrating wire piezometer design and installation in boreholes in Welland, Ontario, to measure uplift pressures on tunnel portal structures for the Ministry of Transportation, Ontario.  Readings were data logged and alarm software developed for notification via a remote communications link.

Pre-bid and final designs of underground excavations for the Western Beaches combined sewer outfall storage tunnel (3 m dia, 4 km long) in Toronto, Ontario.  The project also included excavation of 30-m dia. shafts through rock to tunnel elevation.  Ongoing site visits and consultation during construction.

Design of bar anchor and lining support for two 9-m dia. mine shafts in Wyoming for use as storage bins.

Coordination of tender preparation and pre-bid design of underground support, including large bar anchors and shotcrete for the Nam Ngum III project in Laos (involving a 440-MW underground powerhouse scheme and 10 km of 5-m dia. tunnels).

Project Manager for the Welland River shoreline rehabilitation project, involving the stability assessment of natural, concrete wall and other shoreline types along an unused ship canal and the development of remedial recommendations and cost estimates.

Tunnel inspections and stability reviews for two 4- to 6-m dia. power tunnels in northern Ontario.  Review of existing bar anchor and shotcrete support and design of remedial measures.

***Acres International*** **Senior Geological Engineer** ***1990—1996***

Design of layout and support for an underground powerhouse and water transfer tunnels for the Alto Cachapoal project, Chile, including excavation design and sequencing, bar anchor and shotcrete support design for both drill and blast and TBM excavation.

Site reconnaissance and project layout review for the Upper Marsyangdi hydroelectric project, Nepal.

Review of alkali-aggregate reactivity (AAR) in the concrete of three locks in the St. Lawrence Seaway system, including the preparation of instrumentation plans for long-term monitoring of concrete growth.

Assessment of rock/concrete shear strengths for the stability analyses of a number of concrete gravity structures in northern Ontario.

Review of conceptual plans for the underground storage of low-level radioactive waste in Ontario.  Assisted in the preparation of tenders and contract awards.

Responsible for the long-term automation planning for geotechnical instrumentation at a number of hydroelectric plants in northern Ontario, and the design and installation of remote monitoring systems.

Project Engineer for the Malvern remedial project, involving site preparation for the sorting and storage facilities for 9000 m3 of mildly radioactive contaminated soils. Responsible for detailed final designs, contract preparation, construction supervision and contract closeouts.

Resident Site Engineer during the Stage 3 geotechnical investigations for the Niagara River hydroelectric development. These investigations included the construction of a 600-m long, 3.5-m dia. adit and a 13.5-m dia. trial excavation chamber in the Queenston shale by means of roadheader. The site was affected by high in situ stresses, highly corrosive groundwaters and swelling rock conditions. The program also included various instrumented arrays, measurement of in situ stresses, borehole dilatometer and geophysics testing, and the development of special in situ and laboratory testing for rock swell.

Geotechnical coordination for the definition phase design of the generation facilities for the Niagara River hydroelectric development. These facilities included a proposed 26-m wide powerhouse cavern, transformer gallery, 12.5-m dia. penstock and tailrace tunnels, and associated access tunnels. Design studies included 3D boundary and finite element analyses of the underground complex, and preliminary support design. The work was superseded by the Niagara Diversion Tunnel project.

Coordination of geotechnical and geophysical field investigations for feasibility and final design of a proposed extension to the Owen Falls generating station, Uganda. Responsible for overall review of project seismicity. The project involves a 20-m deep cut in residual soils for a 1-km long power canal and an intake structure on very weak bedrock foundations.

Coordination of investigations for feasibility of siting a health center on an existing landfill in Toronto, Canada. Specific concerns included methane gas control and excavation/redisposal of solid wastes.

***Acres International*** **Geological Engineer** ***1978—1990***

Resident geotechnical representative at the construction site of the 540-MW Chamera hydroelectric generating station in Himachal Pradesh for National Hydro Power Corporation of India. Work included

- supervision of rock excavation, rock support and quality control for 9.5-m dia, 6.5-km long power tunnel, underground powerhouse complex and tailrace tunnel. Installation of 10.5-m long, 52-mm high tensile hollow core bar anchors in crown, and 13-m long, 36-mm dia anchors in walls
- training and transfer of technology to a group of local engineers and geologists
- geotechnical instrumentation.

Planning and supervision of drilling, testing and instrumentation at the Mactaquac generating station, New Brunswick. Investigation of structures and their foundations included determination of concrete characteristics of, and defining the cause of movements within, a concrete gravity intake/spillway and powerhouse. Fieldwork involved a study of concrete cracking and construction joint conditions by borehole photography and ultrasonic methods. Instrumentation installations included tape and borehole extensometers, normal and inverted plumb lines, various deformation and strain gauges, pneumatic piezometers and thermocouples. Concrete tests included direct shear, strength and index properties, thermal properties and both standard and non-standard tests for potential AAR.

Involved in geological investigations for a major project in the Middle East including in situ stress measurements in deep boreholes and core orientation studies.

Development of stereographic projection, statistical analysis and other computer program packages for general use in interpretation of geological field data.

Feasibility site reconnaissance and geologic interpretation of general site conditions for the Sentani hydroelectric project in Irian Jaya, Indonesia. Project involved 4-m dia. tunnels through karstic limestone ridges and 20-m deep channel excavations in weak soils. Evaluation of bids for field explorations.

Detailed dam abutment geologic mapping and assessment for the proposed Granite Canyon hydroelectric development, Yukon.

Development of a computer aided borehole photography interpretation system for use in a foundation investigation program for the spillway at the Limestone generating station, Manitoba.

Prefeasibility site reconnaissance for the Yom-Nan diversion project in Thailand. Responsible for interpretation of general site conditions for determination of project feasibility. Project involved a 25-km long, 8-m dia. tunnel in rock and 48-km of canal excavation in residual soils.

Responsible for geological mapping, core logging, borehole photography and geological interpretation of site conditions for the Upper Salmon hydroelectric development, Newfoundland. Project involved excavations up to 25 m in vertically fissile rock, greater than 5 km of earth-fill dams and dikes up to 25 m high, and overburden excavation for diversion channels. Preparation of data for tenderers and technical specifications for contract purposes. Calculations of rock slope stability and support requirements on site during part of the construction period to review the rock excavations and performance of bar anchors. Responsible for installation of dam instrumentation and supervision of post-impounding monitoring program. Instrumentation included inclinometers, hydrostatic settlement profile gauge, tape extensometer and pneumatic piezometers.

Layout, field supervision and report on an exploratory drilling program related to the stability of a powerhouse rock intake tunnel near Wawa, Ontario.

Responsible for geological mapping, core logging and geological interpretation of site conditions during a major investigation program for a thermal power generating station at Atikokan, Ontario. Project involved dam rehabilitation, construction of rock tunnels, deep excavations in rock and overburden, and heavy structure foundations.

## TECHNICAL PAPERS AND MAJOR PRESENTATIONS

Dam Safety Risk – Canadian and BC Hydro Perspectives, ICOLD-INCA Symposium on Dam Safety for the Americas, Mexico City, October 2016

Next Steps in BC Hydro's Risk Informed Decision Making, keynote presentation at ANCOLD Annual Conference, Brisbane, November 2015

BC Hydro Seismic Hazard Model, presentation at Emergency Preparedness and Business Continuity Conference, Vancouver, November 2014

Reframing Risk Informed Decision Making at BC Hydro, Canadian Dam Association Conference, October 2014 and keynote speech, 2013 HG Acres Seminar, Niagara Falls.

Why Every Owner Needs Risk Informed Decision Making, presentation at CEATI Dam Safety Interest Group meeting, Vancouver, October 2013

Assessment of Extreme Flood Hazard, Series of articles for CDA Newsletter, 2011-2013, that led to the 2013 revision of the CDA Guidelines

Assessment of Shear Resistance for Blasted Rock Foundations, Canadian Dam Association Conference, September 2007.

The Design and Construction of the Shikwamkwa Replacement Dam, Canadian Dam Association Conference, September 2007. (Coauthor)

Accounting for Time-Dependent Deformation in the Niagara Diversion Tunnel Design, Proc. 1st Canada-US Rock Mechanics Symposium, Vancouver, May 2007

Concepts of Shear Resistance and Practical Applications. Dam Engineering, Volume XVI, Issue 3, November 2005. (Coauthor)

Monitoring Sinkhole Development by Detailed Sonar Profiling. Proceedings; Association of State Dam Safety Officials Annual Conference, September 2005, and Proceedings; Canadian Dam Association Conference, October 2005. **Best Paper Award**

The Assessment of Sliding Resistance Beneath Concrete Structures. WaterPower XIII, July 2003. (Coauthor)

Grouting of a Karstic Arch Dam Foundation. 55th Canadian Geotechnical Society Conference, Niagara Falls, Ontario, October 2002. (Coauthor)

A Phased Approach to the Rehabilitation of an Aging Northern Dam. HydroVision 2000 Conference, August 2000. (Coauthor)

Exploratory Adit Program for the Niagara River Hydroelectric Development. 12th Annual Canadian Tunneling Conference, Vancouver, BC, October 1994.

Design of Underground Powerhouse Complex, Niagara River Hydroelectric Development. 45th Canadian Geotechnical Conference, Toronto, Ontario, October, 1992.

Placement and Performance of Impervious Fill Blankets on Slopes. 44th Canadian Geotechnical Conference, Calgary, Alberta, 1991. (Coauthor)

Rock Support for a Large Underground Cavern at Chamera. All India Conference on Underground Engineering, Lucknow, India, February, 1989.

Engineering and Construction Options for the Management of Slow/Late Alkali-Aggregate Reactive Concrete. Proceedings, 16th International Congress on Large Dams, San Francisco, 1988. (Coauthor)

Laser Strain Measurement System. Paper presented at Annual Meeting, Association of Engineering Geologists, Hershey, Pennsylvania, 1978.

The Effect of Sodium Chloride on Water Sorption Characteristics of Rock Aggregate. Bulletin, Association of Engineering Geologists, Vol XIII, No. 3, 1976. (Coauthor)

**LANGUAGES**

English

# Resume Relative to Spillways

**Name:** John Trojanowski, PE

**Nationality:** US Citizen

**Profession:** Consultant Dam Engineering and Hydraulic Structures

**President:** Trojanowski Dam Engineering, Limited

**Qualifications:**

- Over 36 years as a concrete dam, spillway, and outlet works designer.
- Participated in Reclamation's Dam Safety program for 36 years.
- Author of several papers related to spillway failure modes.
- Instructor for the US Army Corps of Engineers' (USACE) Hydraulic Structure Workshop.
- Many years as a Bureau of Reclamation (Reclamation) Dam Safety Advisory Team (DSAT) member.
- Risk Facilitator for Reclamation specializing in concrete dams, spillways, and outlet works since 1995.
- Senior Engineer, Peer Reviewer, Technical Response Team member for many Comprehensive Facility Reviews (CFRs) and Comprehensive Reviews (CRs) for Reclamation.
- Value Planning Team Member to develop the Comprehensive Review process currenty used by Reclamation.
- Consultant for B.C. Hydro for Bennett Dam Spillway chute.
- Team member for the USACE spillway failure mode tool box.
- Member of Bluestone Dam Stilling Basin Design Expert Panel for USACE that included evaluation of spillway hydraulic model studies and hydraulic designs, and spillway drainage designs.
- Participated as a spillway expert for the Bluestone Dam risk analysis.
- Managed 15 to 25 Civil Engineers as the General Manager (GM) for the Reclamation Waterways and Concrete Dams Group.
- Served on several Reclamation Project Management Teams (PMT) for dam safety projects.
- Senior Engineer for Comprehensive Reviews that including:  Stony Gorge, American Falls, Grand Coulee, Hoover, Morrow Point, Crystal, Bartlett, Altus, and Parker Dams.
- Project Manager for several Reclamation Issue Evaluation and Corrective Action Studies including: Parker, Grand Coulee, Morrow Point, Flaming Gorge, Crystal, Minidoka, Hyrum, Gerber, Pueblo, Clear Creek, Navajo, Minidoka (new spillways and post-tensioned tendon stabilizations), Hyrum, Clear Creek, Pueblo, Glen Canyon,
- Project Manager for several Reclamation projects for new or modified dams, including Milltown Hill (Proposed new RCC dam), Kirby Dam (Reconstruction following dam failure), Brantley (new concrete dam), Stewart Mountain Dam Modifications, Spring

Creek Debris Dam Raise, Cold Springs dam modifications, Minidoka Spillway and Powerplant modifications, Theodore Roosevelt (Outlet Works Gate Shaft, Tunnel, and Control Structures).

- Participated on the USACE review team for Gavins Point Dam Spillway.
- Potential Failure Mode Analysis team member for the USACE's John Day Dam.
- Member of a team of experts that reviewed and prepared reports for numerous dam projects for the government of Taiwan.
- Acted as a technical advisor for the National Park Service for the historic Pawtucket Dam.
- Performed hydraulic analyses for several spillways to provide input to Reclamation's Engineering Monograph No. 42 related to cavitation.
- Developed criteria for spillway failure modes used in the USACE and Reclamation Best Practices handbook.

**Experience:**

President: Trojanowski Dam Engineering, Limited since 2015

General Manager/Civil Engineer: Waterways and Concrete Dams Group at the Bureau of Reclamation's Technical Service Center (2010-2014)

Civil Engineer Specializing in Dams and Waterways: U.S. Department of the Interior, Bureau of Reclamation, Denver Colorado from 1978 to 2014 (Retired after 36.5 years)

**Education:**

B.S.C.E. from the University of Colorado in Boulder, Colorado

**Registration:**

Professional Engineer in the State of Colorado

**Papers:**

"Dam Safety Modifications for Clear Creek Dam," Manuscript for ASCE Hydraulics Division, 1993 National Conference on Hydraulic Engineering, July 1993, Author.

"Stabilization of Minidoka Dam Using Epoxy Coated Strand Rock Anchors," ASDSO Dam Safety Conference, Pittsburgh, PA, September 1997, Co-author.

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