NATIONAL TECHNICAL UNIVERSITY OF ATHENS

SCHOOL OF CIVIL ENGINEERING

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AND THE ENVIRONMENT

The Oroville Dam 2017 Spillway Incident

Possible Causes and Solutions

Diploma Thesis

Author:

Aristotelis-Efstathios Koskinas

Supervising Professor:

Demetris Koutsoyiannis, NTUA

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Foreword and Acknowledgements

Chance governs a larger part of our lives than we would like to admit. Even though we incessantly plan for the future, trying to analyze all the possible outcomes and their consequences, only rarely do events occur exactly as they were predicted. And in the aftermath of an unexpected disaster, often people who are complete outsiders to the event attempt to find its causes by interpreting the results. One may consider this to be a negative facet of human nature, but it does have merits. After all, learning from past mistakes is the only way one can prepare for the future.

However, when studying accidents and incidents, it is important to maintain the proper scientific approach and keep an appropriate level-headed tone. Otherwise, we risk becoming conspiracy theorists, seeing patterns and probable causes where there are none, and throwing out wild accusations without any basis. This is nothing but a selfish attempt to prove to ourselves that, if *we* were in control of the situation instead, *we* would have prevented it, and briefly satisfy our insecurities before forgetting about the incident entirely. That is the true negative aspect of our bizarre curiosity related to the misfortunes of our fellow man.

Civil and environmental engineering often feels more like a game of chance rather than the result of a deterministic procedure. Throughout my entire career as an undergraduate student at the National Technical University of Athens, professors bombarded us with examples of spectacular dam failures that led to hundreds of even thousands of deaths, the causes of which weren't apparent until mere days before disaster struck. The accidents at Vajont and Malpasset Dams are the topic of the school's very first lecture given to first year students. Ironically, the very first and very last thing I do at this institute is related to a dam failure. At this point, one may be forgiven to believe that since man-made structures have a finite life expectancy, which can be cut short in a number of unexpected ways, the only winning move is not to build anything ever. But this is a deafeatist's approach.

By chance, I came across a BBC article on February 17th of 2017 (BBC, 2017) related to the Oroville Dam incident. At the time, I hadn't realized the depth of the situation and the processes that led to the subsequent events, so I briefly forgot about it. It would not be until early March that this topic would be brought up again. As I was walking through the university campus one day, contemplating what subject I should choose for my diploma thesis, by chance I bumped into Professor Demetris Koutsoyiannis. When I asked him about his opinion, he immediately mentioned the spillway incident, pointing out its uniqueness as a dam failure that occurred seemingly without precedent under natural operating conditions, which posed several questions on how to properly operate and inspect aging structures. For this reason, he is the first person I must thank, both for supervising my diploma thesis and for his constant willingness to assist me whenever I had any questions related to the topic.

Shortly after agreeing on the thesis subject with Prof. Koutsoyiannis, David Hagen wrote *Will the Oroville Dam survive the ARkStorm?* (Hagen, 2017), a very interesting article that explained the Oroville spillway incident from a much more scientific point of view than that of local news outlets at the time, also citing Prof. Koutsoyiannis' papers on climate persistence. So, by chance, only weeks after I selected the Oroville Dam spillway incident as a subject, an article emerged that mentioned our own institute by name, thus linking it to the related discussion. At this point, I must express my gratitude to Mr. Hagen, initially for writing this article, but primarily for his personal interest in my thesis, and for leading me to several articles, official reports and public discussion forums that provided the foundation upon which this thesis would be created. Of course, I must also thank Judith Curry for hosting Mr. Hagen's work on her website, and also for expressing her own interest in my thesis.

Aside from Prof. Koutsoyiannis, I must also personally thank ITIA research team members Panayiotis Dimitriadis and Theano (Any) Iliopoulou. They provided me with critical advice that was paramount to the creation of the thesis and the validation of its results, and were not afraid of working overtime just for the sake of helping me and other students understand the vast concepts of science and engineering. It was with them that I would embark on a journey to Vienna for the 2017 European Geosciences Union General Assembly. Hours upon hours were spent on a large venture that encompassed multiple aspects of engineering and the creation of sustainable energy solutions (Daniil, et al., 2017). Approximately 32 students and faculty members in total worked on this project, and I would like to thank everyone, especially mentioning my good friends George Pouliasis and Eleni Zacharopoulou. It was with them that I collaborated on a specific aspect of this assignment. They helped me both directly in this thesis by contributing in the creation of the MATLAB scripts used for data analysis, and indirectly by providing consulting and moral support whenever I required it. I would also like to thank Prof. Panos Papanicolaou for helping me create the 1-D water surface profile models of the Oroville Dam main spillway chute in Excel, thus saving me from the nightmare of HEC-RAS software limitations.

Finally, none of this would have been possible without the help of my immediate friends and family. I would especially like to thank my parents Spyridon Koskinas and Anne-Frances Sordinas, for their continuous support and guidance which helped me overcome the trials and tribulations of everyday life. And of course, I must also express my gratitude to my sister Marianna Koskina for believing in me throughout the years, and whose love of history and literature provided me with the introduction's opening quote.

Abstract

The subject of this thesis is the 2017 Oroville Dam spillway incident. This event is yet another example of the severe problems the United States has with maintaining its large infrastructure. However, in order to better understand how events unfolded, it is first necessary to conduct a detailed analysis of Oroville Dam and the basic elements of its location, the Feather River Basin.

An assessment of the hydroclimatic characteristics of the area reveals it to have a Mediterranean climate, indicated by heavy precipitation during the winter months, producing floods during the spring, followed by almost completely dry summers. From a geological standpoint, the area near Oroville Dam contains metavolcanic rock, which is of adequate hardness, but it is also significantly weathered, especially near the ground surface.

This thesis also contains a summary of various Oroville Dam design elements, as well as a full history of its construction. This analysis reveals hidden clues that help identify the causes of the 2017 incident. Most significantly, design criteria for the main and emergency spillways appear much more relaxed than those of the main structure.

Next, a study of the previous significant floods that occurred at Oroville Dam is conducted. This reveals that reservoir levels were much higher during the 2017 incident compared to other events, which indicates a need to lower the minimum flood control elevation.

Futhermore, this thesis includes an extensive timeline of the 2017 incident events, including the damages to Oroville Dam's main spillway chute and area downstream of the emergency spillway. After further research, initial cause of the main spillway failure is defined as concrete chute floor slab uplift, caused due to faults in the drain system below it.

In addition, perusal of previous official inspection reports reveals that under current practice standards, if a comparable incident occurs again, its indications are unlikely to be detected in time.

Finally, recommendations are made in order to avoid similar events from happening in the future. For Oroville Dam, this means lowering the minimum flood control elevation level and creating a fully armored concrete emergency spillway. In the short term, informal inspections by the authorities that operate large structures in the United States can discover faults before they turn into accidents. However, a more long-term plan to effectively repair and maintain the country's existing infrastructure needs to be put into action immediately.

Εκτεταμένη Περίληψη στα Ελληνικά (Executive Summary in Greek)

Το φράγμα Oroville είναι ένα γεώφραγμα ύψους 234.7 μέτρων, χτισμένο στους πρόποδες της Sierra Nevada, στην κοιλάδα του ποταμού Feather, στην κοινότητα Butte της πολιτείας της Καλιφόρνια. Είναι το ψηλότερο φράγμα στις Ηνωμένες Πολιτείες της Αμερικής και το πέμπτο ψηλότερο φράγμα στον κόσμο. Ένα πλήθος παραποτάμων του ποταμού Feather ρέει μέσα στον ταμιευτήρα του φράγματος, και από εκεί καταλήγουν στον κύριο ποταμό κατάντη. Έπειτα, αυτός ο ποταμός αποτελεί τμήμα ενός μεγαλύτερου δικτύου ποταμών, οι οποίοι καταλήγουν στον ποταμό Sacramento. Αυτός με τη σειρά του εκβάλει στον Ειρηνικό Ωκεανό, κοντά στην πόλη του San Fransisco.

Η λέξη «Oroville» έχει ισπανικές και γαλλικές ρίζες και σημαίνει «χρυσή πόλη». Το Oroville απέκτησε αυτό το όνομα όταν ανακαλύφθηκε κοίτασμα χρυσού στην περιοχή το 1848, το οποίο γρήγορα εξαντλήθηκε με την επέλαση των χρυσοθήρων. Μετά από αυτά τα γεγονότα, οι κάτοικοι της περιοχής στράφηκαν στην καλλιέργεια της γύρω εύφορης γης για να καλύψουν τις ανάγκες τους. Από τότε διαπιστώθηκε η μεγάλη ζήτηση της περιοχής για νερό με σκοπό τη χρήση του για άρδευση. Οι ανάγκες αυτές έγιναν πιο σπουδαίες από ποτέ μετά τη λήξη του Β’ Παγκοσμίου Πολέμου, όπου πλήθος φτωχών Αμερικανών πολιτών μετακόμισαν στις εύφορες κοιλάδες της Καλιφόρνια, αναζητώντας ένα καλύτερο μέλλον για τις οικογένειές τους.

Τότε, η πολιτεία πρότεινε ένα σχέδιο εκτροπής του ποταμού Feather, με σκοπό την αποταμίευση της περίσσειας νερού στη Βόρεια Καλιφόρνια και τη διάθεσή του στις ξηρές εκτάσεις του Νότου, το οποίο εμπεριείχε την κατασκευή μεγάλων έργων όπως το φράγμα Oroville. Το σχέδιο αυτό αντιμετώπισε σκληρή κριτική από τους κατοίκους του Βορρά, οι οποίοι θεωρούσαν ότι το νερό της περιοχής τους ανήκε δικαιωματικά και δεν ήθελαν να διοχετευτεί ούτε σταγόνα στο Νότο. Εν τέλει, μετά μια καταστροφική πλημμύρα στην κοιλάδα Feather το 1955 με πλήθος νεκρών και εκατομμύρια δολάρια σε ζημιές, το σχέδιο εκτροπής πέρασε ως νομοσχέδιο στο ψηφοδέλτιο του 1960, στις ίδιες εκλογές που όρισαν τον Τζων Κέννεντυ νέο πρόεδρο των ΗΠΑ.

Η κατασκευή του φράγματος Oroville ήταν το πιο ακριβό και το μεγαλύτερο σε έκταση έργο στις ΗΠΑ μέχρι εκείνη την εποχή. Κόστισε συνολικά 438 εκατομμύρια δολάρια όταν ολοκληρώθηκε το 1968, που σε σημερινά χρήματα θα ήταν ποσότητα ίση με 3 δισεκατομμύρια. Το κύριο φράγμα είναι διαζωνισμένο χωμάτινο με κεκλιμένο αργιλικό πυρήνα. Ο τύπος αυτός επιλέχθηκε λόγω του πλήθους κατάλληλου υλικού που είχε περισσέψει στην κοιλάδα από την εποχή των χρυσοθήρων, και ο κεκλιμένος πυρήνας επιλέχθηκε ώστε να αντιμετωπιστούν καλύτερα σεισμικές διαταραχές, αλλά και λόγω της σχετικής έλλειψης αδιαπέρατου υλικού στη γύρω περιοχή.

Η διπλωματική εργασία περιέχει επίσης και μια πλήρη περιγραφή διάφορων φυσικών χαρακτηριστικών της υδρολογικής λεκάνης του ποταμού Feather.

Πρόκειται για μια ως επί το πλείστον ορεινή λεκάνη, με περίπου 55% της έκτασής της να βρίσκεται σε υψόμετρο άνω των 1.500 μέτρων, που είναι το νοητό επίπεδο χιονόπτωσης της περιοχής. Ανάντη του φράγματος Oroville, υπάρχει πλήθος μικρότερων αναρρυθμιστικών ταμιευτήρων που καλύπτουν παρόμοιες ανάγκες.

Από γεωλογικής άποψης, στην περιοχή του φράγματος κυριαρχούν μεταμορφωμένα πετρώματα όπως ο αμφιβολίτης, επικαλυπτόμενα από αργίλους και ιλύες που προέρχονται από αποθέσεις του ποταμού. Η περιοχή δεν είναι ιδιαίτερα σεισμογενής, ιδίως αν συγκριθεί με τη σεισμογένεια της Καλιφόρνια γενικότερα.

Το κλίμα της λεκάνης του ποταμού Feather είναι εύκρατο μεσογειακό (κατηγορία Köppen Csa) όπως ακριβώς αυτό της Αθήνας. Υδρολογικά, χαρακτηρίζεται από εξαιρετικά ξηρά καλοκαίρια, με σχεδόν μηδενικές βροχές, και εξαιρετικά υγρούς χειμώνες, με το μεγαλύτερο ποσοστό της ετήσιας βροχής να πέφτει το Δεκέμβρη και το Γενάρη. Μια ανάλυση παροχών των παραποτάμων του Feather ανάντη του φράγματος δείχνει ότι το μεγαλύτερο ποσοστό των ετήσιων απορροών δημιουργείται τον Απρίλη, με το λιώσιμο του χιονιού στις κορυφές των γύρω βουνών.

Το ιστορικό των πλημμύρων στις περιοχές κατάντη της λεκάνης ξεκινά από τις αρχές του $20^{ου}$ αιώνα, με καταγεγραμμένες παροχές να υπάρχουν μέχρι και για το υδρολογικό έτος 1901-02. Από τότε είχε καταγραφεί η ανάγκη για την κατασκευή αντιπλημμυρικό έργου, όμως καλύφθηκε τελικά το 1968 με την κατασκευή του φράγματος Oroville.

Εικόνα 1. Ζημιές στην πόλη του Oroville μετά την πλημμύρα του Μάρτιου 1907.

Από κατασκευαστικής άποψης, το φράγμα Oroville πρόκειται για μία χωμάτινη διαζωνισμένη κατασκευή, ύψους 235 μέτρων. Το κύριο φράγμα έχει συνολικό όγκο 61 εκατομμυρίων κυβικών.

Εικόνα 2. Αεροφωτογραφία του φράγματος Oroville (2008).

Το φράγμα αυτό έχει δύο ανεξάρτητους υπερχειλιστές. Ο πρώτος, κύριος υπερχειλιστής αποτελείται από μια ανεπένδυτη ανοιχτή διατομή εισόδου, μια πύλη με οκτώ περιστρεφόμενα θυροφράγματα, που επιτρέπουν τον έλεγχο της παροχής εξόδου, και έναν αγωγό από επενδυμένο μπετό, ορθογωνικής διατομής, μήκους 914 μέτρων. Ο δεύτερος πρόκειται για έναν υπερχειλιστή «έκτατης ανάγκης», που είναι μια απλή κατασκευή τύπου ogee από ανεπένδυτο μπετό, χωρίς κάποια διαμόρφωση κατάντη, που σημαίνει ότι αν μπει σε χρήση, οι εκροές ρέουν πάνω στο φυσικό έδαφος.

Επίσης, το φράγμα Oroville διαθέτει σταθμό παραγωγής υδροηλεκτρικής ενέργειας, αποτελούμενη από 6 στροβίλους τύπου Francis, με συνολική εκτιμώμενη ισχύ στα 679 MW. Υπάρχει επίσης και δυνατότητα αντλησοταμίευσης.

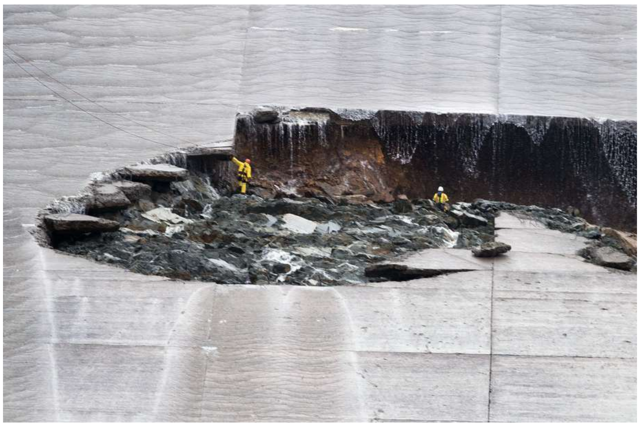
Για λόγους πληρότητας, στο πλήρες κείμενο καταγράφονται διάφορες κατασκευαστικές λεπτομέρειες του φράγματος Oroville, πολλές εκ των οποίων συνέβαλαν στα αίτια και τα αποτελέσματα του ατυχήματος του Φεβρουαρίου 2017. Ενδεικτικά, ενώ οι υπερχειλιστές και το κύριο φράγμα κατασκευάτηκαν πάνω σε άρρηκτο πέτρωμα μετά από εκτενείς εκσκαφές, η γενικότερη βραχόμαζα της περιοχής είναι λίγο έως πολύ αποσαθρωμένη. Δεν επενδύθηκε με μπετό η περιοχή κατάντη του υπερχειλιστή έκτακτης ανάγκης, ούτε έχει αφαιρεθεί ποτέ η φυτοκάλυψη, με τη δικαιολογία του ότι θα χρησιμοποιηθεί σπάνια ή και ποτέ.

Πριν τα γεγονότα του ατυχήματος, δύο μεγάλες πλημμύρες έχουν συμβεί στην περιοχή το 1986 και το 1997, όμως και οι δύο ήταν εντός των κριτηρίων σχεδιασμού και δεν πρόεκυψε κανένα πρόβλημα κατάντη. Ωστόσο, παρότι η πλημμύρα του Φεβρουαρίου 2017 ήταν ακόμη χαμηλότερη, προκάλεσε εκτεταμένες ζημιές στο φράγμα.

Στις 6 Φεβρουαρίου, εργάτες στο φράγμα Oroville παρατήρησαν μια μεγάλη τρύπα στον πυθμένα του αγωγού του κύριου υπερχειλιστή.

Εικόνα 3. Φεβρουάριος 6, 2017. Πρώτες ζημιές στον αγωγό του κύριου υπερχειλιστή.

Μια δεύτερη φωτογραφία βοηθάει στην εκτίμηση του εύρους της ζημιάς.



Εικόνα 4. Φεβρουάριος 6, 2017. Εργάτες εξετάζουν από κοντά το εύρος της ζημιάς.

Για να αποφευχθούν περαιτέρω ζημιές στον κύριο υπερχειλιστή, αποφασίσθηκε να μπει σε λειτουργία ο εναλλακτικός. Ωστόσο, γρήγορα χρειάστηκε να διακοπεί η λειτουργία αυτή, καθώς οι ροές κατάντη του διάβρωσαν γρήγορα το έδαφος από κάτω, κινδυνεύοντας να φτάσουν κάτω από τον υπερχειλιστή έκτακτης ανάγκης και να τον καταστρέψουν. Τότε, κρίθηκε απαραίτητη η χρήση του τρυπημένου κύριου υπερχειλιστή, ο οποίος μπόρεσε να αντέξει μεγάλες εκροές χωρίς να καταστραφεί ολοσχερώς.

Εικόνα 5. Φεβρουάριος 11, 2017. Ο υπερχειλιστής έκτακτης ανάγκης μπαίνει σε λειτουργία.

Φοβούμενες μια πιθανή πλημμύρα στην κοιλάδα κατάντη του φράγματος, οι αρχές αναγκάστηκαν να εκκενώσουν τη γύρω περιοχή, και συνολικά πάνω από 180.000 κάτοικοι αναγκάστηκαν να εγκαταλείψουν προσωρινά τα σπίτια τους. Ευτυχώς, δεν υπήρχαν ανθρώπινα θύματα, αλλά προξενήθηκαν εκτενείς οικολογικές και οικονομικές ζημιές. Παρακάτω φαίνονται οι ζημιές που προξενήθηκαν και στους δύο υπερχειλιστές.

Εικόνα 6. Φεβρουάριος 13, 2017. Ζημιές στον εναλλακτικό υπερχειλιστή του φράγματος Oroville.

Εικόνα 7. Φεβρουάριος 27, 2017. Ζημιές στον κύριο υπερχειλιστή του φράγματος Oroville.

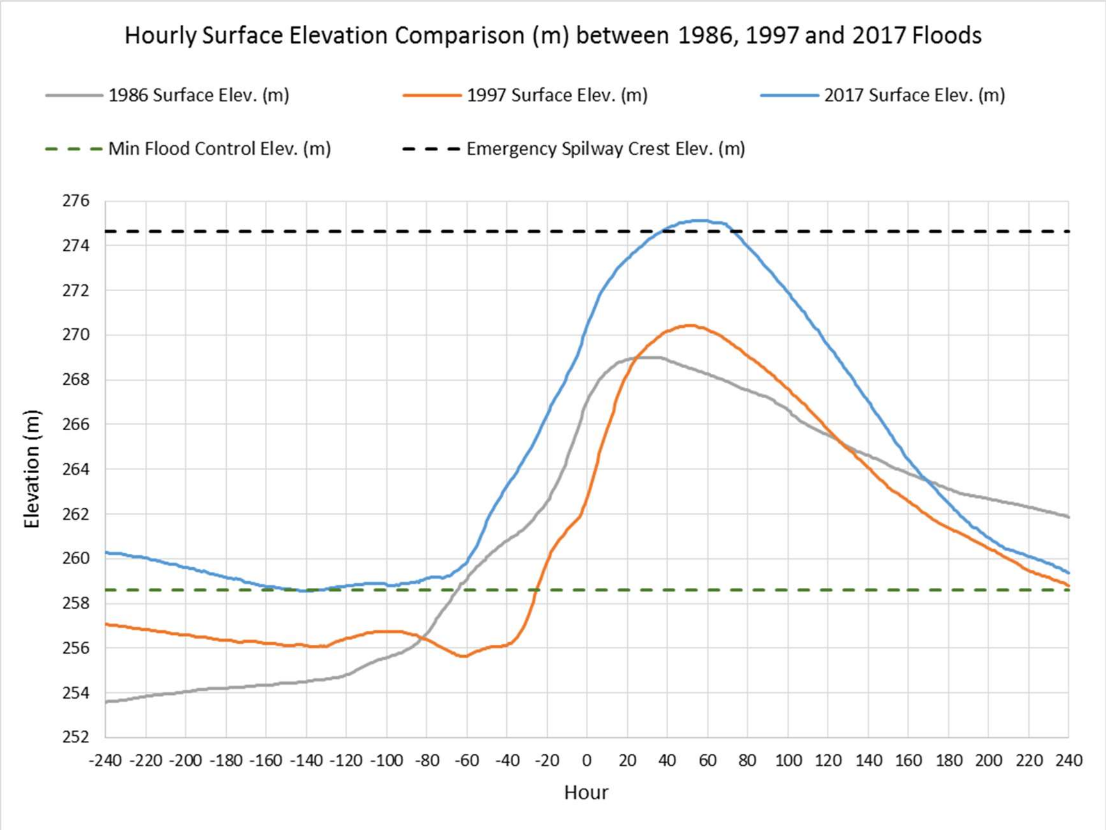
Μετά το ατύχημα, η τοπική υπεύθυνη αρχή (California Department of Water Resources) αμέσως ξεκίνησε επισκευές στον κύριο υπερχειλιστή και πρότεινε επανξέταση της κατασκευής του εναλλακτικού. Εκτιμάται ότι μέχρι το 2018 οι επισκευές θα έχουν ολοκληρωθεί, ενώ ο κύριος υπερχειλιστής θα είναι ξανά έτοιμος για χρήση.

Σαν κύρια αίτια του προβλήματος, τελικά επιλέγονται τα εξής:

Πρώτον, η αρχική τρύπα στον αγωγό του κύριο υπερχειλιστή προξενήθηκε από μάζα νερού από κάτω του, η οποία είχε περάσει μέσα από κενά στο αποσαθρωμένο πέτρωμα της θεμελίωσης και ανασήκωσε τις σκυροδετημένες πλάκες του πυθμένα. Αυτό επιβεβαιώθηκε μετά από εκτενή μελέτη της βιβλιογραφίας, την κατασκευή μοντέλου του αγωγού και την ανάλυση φωτογραφιών λίγο πριν του συμβάντος.

Εικόνα 8. Φωτογραφίες του αγωγού του κύριου υπερχειλιστή του φράγματος Oroville. Η αριστερή λήφθηκε 11 Ιανουαρίου και η δεξιά 27 Ιανουαρίου. Δεξιά φαίνονται οι πρώτες ενδείξεις της ανασήκωσης των πλακών του πυθμένα.

Σαν δεύτερο κύριο αίτιο του ατυχήματος αναφέρεται η στάθμη του ταμιευτήρα αμέσως πριν το συμβάν. Παρότι ήταν ακριβώς στο σχεδιασμένο επίπεδο κατώτατης στάθμης πλημμύρας, ήταν σε πολύ υψηλότερο επίπεδο από αυτό λίγο πριν τις δύο προηγούμενες μεγάλες πλημμύρες του 1986 και 1997, όπως φαίνεται παρακάτω.



Σχήμα 1. Σύγκριση των σταθμών του ταμιευτήρα του φράγματος Oroville δέκα μέρες πριν και μετά τις πλημμύρες των ετών 1986, 1997 και 2017.

Επιπλέον, μετά από εκτενή μελέτη της βιβλιογραφίας, προέκυψε ένα λάθος στην εκτίμηση της κατώτατης στάθμης πλημμύρας σχεδιασμού όταν ήταν ακόμα υπό κατασκευή το φράγμα.

Σαν τρίτο αίτιο αναφέρονται ελλείψεις στις καθιερωμένες αξιολογήσεις του φράγματος. Όπως έδειξε το ατύχημα αυτό, πιθανές ατέλειες μπορούν να προκαλέσουν μεγάλες ζημιές μέσα σε πολύ μικρό χρονικό διάστημα, ιδίως σε μια παλιά κατασκευή όπως το φράγμα Oroville.

Μετά από περαιτέρω ανάλυση, προτείνονται οι παρακάτω λύσεις:

- Πτώση της κατώτατης στάθμης πλημμύρας στα **255** μέτρα. Αυτή είναι η στάθμη που προκύπτει μετά από τη διόρθωση της προαναφερθείσας σχεδιαστικής έλλειψης. Μετά από μελέτη της παροχετευτικότητας του κύριου υπερχειλιστή, διαπιστώνεται ότι το επίπεδο αυτό είναι εφικτό να διατηρηθεί.
- Η κατασκευή του νέου κύριου υπερχειλιστή γίνεται σύμφωνα με όλα τα απαραίτητα σχεδιαστικά πρότυπα και κατασκευάζεται με εκπληκτική ταχύτητα.

Ωστόσο, περαιτέρω ανάλυση δείχνει ότι η κατασκευή αυτή από μόνη της μπορεί να παραλάβει μόνο μέχρι και την πλημμύρα με χρόνο επαναφοράς 500 ετών. Για αυτό, κρίνεται απαραίτητη μια διαμόρφωση αγωγού κατάντη του εναλλακτικού υπερχειλιστή εξ'ολοκλήρου από μπετό, ούτως ώστε το φράγμα Oroville να μπορέσει να παραλάβει την πλημμύρα 10.000 ετών χωρίς να πάθει ζημιές.

- Τέλος, προτείνεται η εβδομαδιαία διεξαγωγή ανεπίσημων αξιολογήσεων των κατασκευών από την αρμόδια αρχή που λειτουργεί το φράγμα, μόνο και μόνο για να διαπιστωθούν οι ενδείξεις τέτοιων μεγάλων ζημιών πριν εξελιχθούν σε ατυχήματα.

Κλείνοντας, αναφέρεται ότι το ατύχημα στο φράγμα Oroville αποτελεί έμβλημα ενός μεγαλύτερου προβλήματος διαχείρισης των μεγάλων έργων στις ΗΠΑ. Πρέπει να διαμορφωθεί μια μακρπρόθεσμη λύση του προβλήματος το συντομότερο, διότι σε διαφορετική περίπτωση τέτοια ατυχήματα θα πάψουν να είναι μεμονωμένα περιστατικά, και το επόμενο μπορεί να στοιχίσει ανθρώπινες ζωές. Η διατήρηση μεγάλων έργων είναι μια μη ιδιαίτερα δημοφιλής διαδικασία, καθώς όταν γίνεται σωστά, πρακτικά δε συμβαίνει τίποτα. Όμως κάποιος πρέπει να την κάνει.

1. Introduction - History of Events Leading up to Oroville Dam's Construction

"Life can only be understood backwards; but it must be lived forwards."

— Søren Kierkegaard

The Oroville Dam is a 234.7 m (770 ft.) tall earth-fill dam. Built in 1968, it is the tallest dam in the United States of America and the fifth tallest earth-fill dam in the world. It is located on the foothills of the Sierra Nevada, on the Feather River, in Butte County, California. In the south, near Marysville and Yuba City, the Yuba River flows into the Feather River, which in turn flows into the Sacramento River, leading up to the city of Sacramento, and finally spilling into the Pacific Ocean near the San Francisco Bay. (California Department of Water Resources, 2017)

Map 1. Map of the Feather River and its tributaries. Source: Shannon1 (2017).

Directly below the dam, on the banks of the Feather River where it flows into the base of the Sacramento Valley, lies Oroville, a city of approximately 20 000 residents. The name Oroville derives from the Spanish term "Oro" which means gold, and "Ville", which indicates a city. Originally called Ophir City, it gained this new identity after gold was discovered at Bidwell's Bar in 1848, near the center of what is now Lake Oroville. This location was one the first gold mining camps in California, but was quickly depleted of its resources during a mining rush in the years 1856 to 1857. Following these events, the miners abandoned Bidwell's Bar and moved to Oroville. The entire area was subsequently flooded in 1968 upon the completion of Oroville Dam's construction. (Miller, 1978)

After the end of the California Gold Rush, the residents of California's Central Valley mainly turned to agriculture as a means of economic growth, starting with cattle ranching. However, the severe drought of 1863-1864 resulted in the loss of most cattle in California. This fact, combined with a population increase in the area and the development of railroads after 1869 made farming the primary form of agriculture in the area and led to a large demand of water for irrigation purposes.

In order to supply water for the Central Valley region, various studies of local water resources were conducted. In 1873, President Ulysses S. Grant commissioned an investigation by Colonel B. S. Alexander of the U.S. Army Corps of Engineers. The state of California also initiated a project of its own in 1878, starting a State Engineer office under William Hammond Hall. This project included a detailed assessment of several aspects of water resources systems; including drainage and river channel investigations, the creation of large scale irrigation maps, and the installation of gages in order to record geographic, geologic, and hydrologic data.

Several years later, in 1919, Lt. Robert B. Marshall of the U.S. Geological Survey formulated a plan for a water development project spanning the entirety of California. Lt. Marshall's idea was centered on using excess water from the Sacramento basins to supply the arid San Joaquin area to the south.

After further research, the plan was finally prepared to be put into action in 1931, under State Engineer Edward Hyatt. Legislature passed the Central Valley Act of 1933, which authorized $170 million to implement the Central Valley Project. Unfortunately, at that time the Great Depression was still in full effect, and the plan was met with strong opposition, mainly from affected parties in the Sacramento Valley who did not want to give any water to the southern regions. Thus, the plan was effectively put on hold until the end of World War II.

After 1945, California was hit by a second "Gold Rush". It was not a gold rush in the literal sense however; having gained a reputation from the earlier days as the "Golden State", people flocked to the Central Valley for its Mediterranean climate and the economic opportunities it could provide. This second large increase in population made the need to supply the local water demand more prominent than ever before.

The state of California immediately launched an investigation of water resources, resulting in the publications of several bulletins, analyzing the existing measurements of precipitation, stream flow, flood frequency, and water quality data. In 1951, the US Bureau of Reclamations looked further into the possibility of a water resources management system that included transferring water between multiple basins. As a part of this project, then California State Engineer A.D. Edmonston proposed damming the Feather River, initiating the Feather River Project, a predecessor of the current California State Water Project (SWP). It was this plan that led to the beginning of Oroville Dam's construction.

However, there was still vehement opposition against this venture. Aside from the previously mentioned concerns from the residents of the Sacramento Valley, people in the southern San Joaquin regions were also worried that the North might later revoke the rights to accessing the water supply. Meanwhile, people living in the San Francisco Bay area feared that their waterways would be flooded and requested assurances that these facilities would be protected. Furthermore, critics of the project believed that the plan was too expensive to realistically achieve. Special committees met with the intent of reach an agreement on all sides, but failed to do so. In the end, the SWP was authorized through the Burns-Porter Act, also known as the California Water Resources Development Bond Act, or Proposition One. This act was placed on the November 1960 ballot and was approved by a very slim margin, rejected by all northern counties except Butte, the area where Oroville Dam would later be built. This allowed construction of the Oroville Dam and its facilities to officially commence.

Yet groundbreaking at the dam's site had already begun since 1957. This was mainly due to the catastrophic floods that occurred in late 1955 to early 1956, which devastated Northern and Central California, including 64 registered deaths and $200 million in property damage. These events led to immediate countermeasures by the State, passing an emergency appropriation of $25 million. These funds covered the costs of initial preparations, including two tunnels relocating the Western Pacific Railroad in order to clear the Oroville Dam site. In addition, in 1956 the State authorized the preparation of final design plans and specifications for the dam. (California Department of Water Resources, 2008); (California Deparment of Water Resources, 1974)

Initial design plans included a gravity, straight-buttress, multiple arch, or arch-buttress concrete dam. However, geologic and construction materials investigations in the area proved the existence of a large amount of dredger tailings, left from gold miners from the 1800's. These included a gravel-type material which, at the time, was deemed perfect for the pervious shells of an earth-fill dam. Analysis showed that these tailings could be used to create an embankment dam at the same cost as a concrete structure, and further research unearthed material near the tailings which could be used for an impervious core as well. The final deciding factor concerning the type of dam that would be constructed were the dam's foundations. Since a concrete structure requires much more extensive foundations, plans for it were ultimately dropped in favor of an earth-fill structure with an inclined core.

Concerning transportation of the borrow materials to the dam site, the solution applied by the dam construction's winning contractor was to utilize remaining railways left off from relocating the Western Pacific Railroad to transfer materials.

After the approval of the Burns-Porter Act in November 1960, construction of the Oroville dam officially began in the summer of 1961, with the award of a contract for building the first of two required diversion tunnels. A second contract for the construction of the main dam and the other diversion tunnel was awarded in 1962. Remaining facilities, such as the spillway, reservoir clearing and saddle dams were accomplished by separate contracts up to four years later (California Deparment of Water Resources, 1974).

2. Feather River Basin Characteristics

2.1 General Description

Feather River Basin lies between the north end of the Sierra Nevada range and the east side of the Sacramento River Valley. It is bounded by Mt. Lassen to the northwest, and by the Diamond Mountains to the northeast. Elevations of the Feather River begin at 3,190 m (10,466 ft.) at Mt. Lassen's peak and end at around 274 m (900 ft.) at Oroville Dam. Around 55 percent of the area is above 5000 ft. (1,524 m), and only 7 percent is below 2000 ft. (609.6 m) (US Army Corps of Engineers, 1970). Feather River's upper reaches branch into several forks: North and Middle Fork which extend up to the east of Mt. Lassen, and West Branch and South Fork lie on the western slopes of the Sierra Nevada. The general flow direction of these streams is south or southwestern, and they all converge at the foothills of the Feather River Canyon just above the location of Oroville Dam. Another important stream is the East Branch, a tributary of the North Fork which ends near Belgen. The steep sloping banks of the North and South Forks have been extensively engineered for hydropower generation purposes: there are several smaller dams, levees, reservoirs, tunnels, and canals that offer a small amount of streamflow regulation above Oroville Dam. Figure 1 below displays a graph of the main reservoirs in the North Fork Feather River, provided by PG&E.

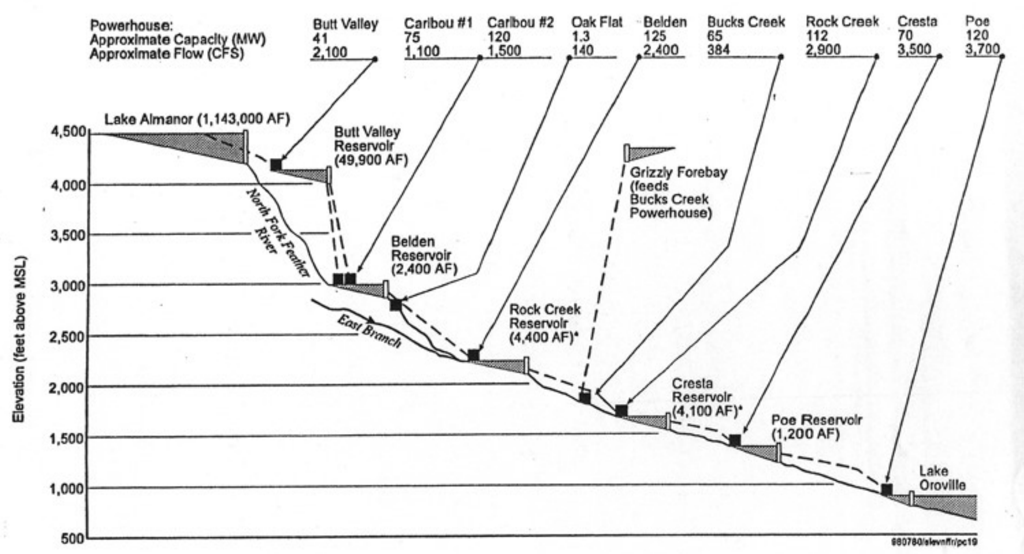


Figure 1. Hydroelectric development in the North Fork Feather River drainage. Source: PG&E (2002).

Table 1 below contains the names and storage capacities of the largest reservoirs in the Feather River basin (DWR, 2004).

Table 1. Largest reservoirs in the Feather River Basin. Source: DWR (2004).

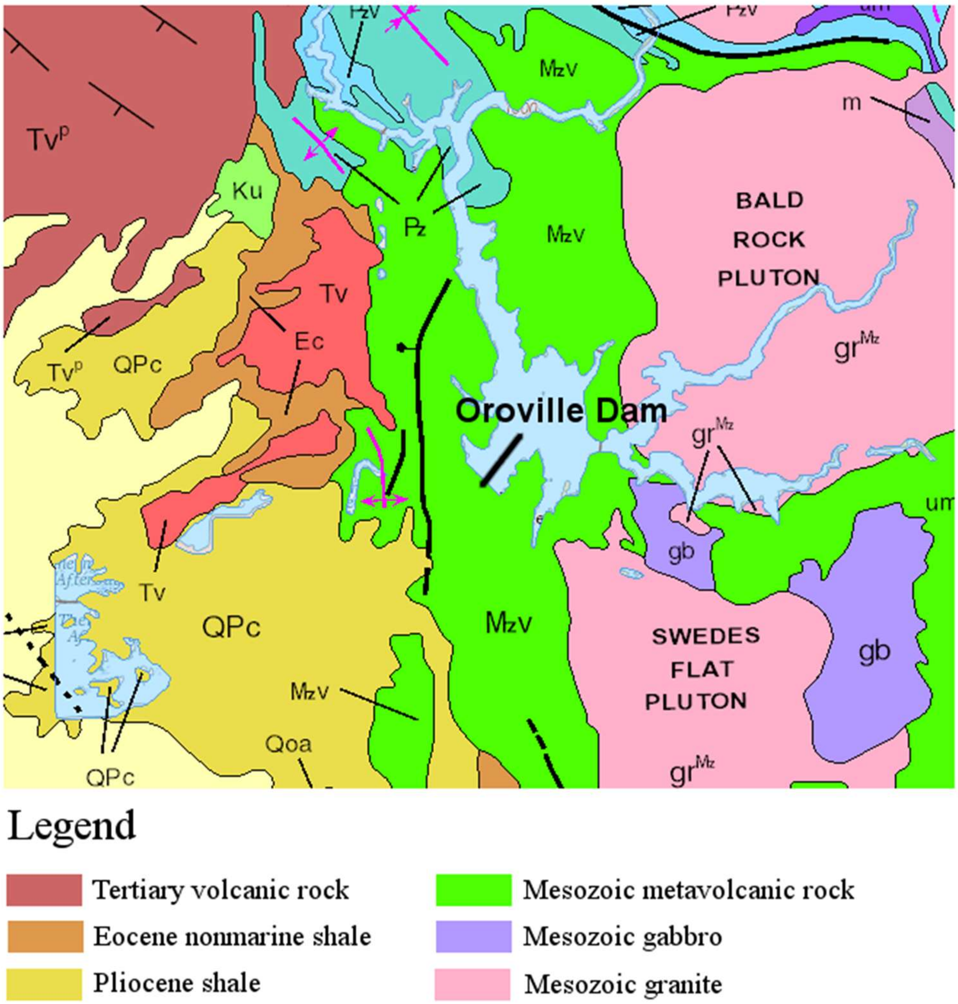
Reservoir Name	Reservoir Storage Capacity (AF)	Reservoir Storage Capacity (hm^3)
Mountain Meadows	7,800	9.62
Lake Almanor	1,308,000	1613.39
Butt Valley	49,700	61.30
Bucks Lake	101,900	125.69
Antelope Valley	22,600	27.88
Frenchman Lake	55,500	68.46
Lake Davis	84,400	104.11
Little Grass Valley	74,400	91.77
Sly Creek	56,200	69.32
Total	1,760,500	2171.54

Vegetation in the Feather River Basin is comprised mostly of coniferous trees, including the Lassen National Forest in the northwest and the Plumas National Forest in the southeast. Heavy timber growth in the westernmost regions is replaced by a sparse cover of shrubs on the eastern slopes where the basin meets semi-arid valleys.

The city of Oroville lies just six miles underneath the Oroville Dam. After leaving the mountains, the Feather River then flows into the plains of the Sacramento valley. This factor, combined with the richness of the soil in the vicinity makes these lands ideal for agriculture. Up above on the basin, however, the area is less rich, and accordingly more sparsely populated. The largest towns directly within the basin are Chester, Quincy, and Portola, with a population of approximately 2000 each (US Census Bureau, 2017). These numbers have been slowly but steadily declining since the beginning of the 21st century.

2.2 Geology and Seismicity

The Feather River Basin is partially located within the Sierra Nevada, characterized by a geological transition between the northernmost sections containing hard, metamorphosed volcanic rock, and the southern areas which primarily consist of sedimentary formations, overlapping a granite core (Durrell, 1987). According to (California Department of Water Resources, 1977), the origin of the metamorphic rock is believed to be from the Paleozoic era, when the area of what is now the Sierra Nevada was the bottom of a prehistoric sea. In Mesozoic time, granitic magma was introduced to the area through a north-trending synclinal trough and a series of intrusions, which are believed to be no longer active. During the orogeny of the Nevada area, this granite was uplifted, and subsequent erosion caused sedimentary rocks that had accumulated since the late Triassic and Jurassic periods to mix with this volcanic material. Various volcanic events occurred in the Oligocene, Miocene, and Pliocene eras, tilting the northern Sierra Nevada westward, then halted. Finally, significant erosion during the Quaternary period created the landscape observed today.



Map 2. Geological map of the area around Oroville Dam. Adapted from Jennings, et. al. (1977)

Studying the geology of the Oroville area is important for discovering how exactly various formations affect the Feather River basin's surface runoff and soil permeability. Documented results of geological studies in the vicinity (California Deparment of Water Resources, 1974); (California Department of Water Resources, 1977) revealed the area immediately in and around Lake Oroville to be comprised mostly of what is called the "Bedrock Series". This consists mostly of metavolcanic and pyroclastic rock, such as amphibolite. Above this bedrock lie various younger sedimentary rocks such as shales, dolomites, Quaternary alluvium, playas, terraces, glacial till and moraines, and finally various marine and non-marine sediments (Jennings, Strand, & Rogers, 1977). According to (Freeze & Cherry, 1979), the volcanic formations in the northernmost sections of the Feather River Basin are the most permeable, and thus transmit a large amount of ground water to streams. On the other hand, the sedimentary formations in the southern region contribute water through surface runoff and subsurface flow as well as from ground water.

In general, the Feather River Basin is considered an area of low seismicity, as earthquakes happen about half as often as the California State average. However, on a few occasions, the area near Oroville Dam has been affected by them. The earliest recorded significant seismic activity occurred in the Mohawk Valley during 1875, 40 miles to the east of Oroville. This area was uninhabited at the time, so no damage was reported, yet the area around the dam was assigned a modified Mercalli seismic intensity scale of V, which indicates a moderate earthquake (California Deparment of Water Resources, 1974); (USGS, 2016). The next recorded significant seismic activity happened in 1940, with a magnitude of 5.7 on the Richter scale. The epicenter of this event was located about 50 kilometers (31 miles) north of Oroville Dam (California Department of Water Resources, 1977). This was the last important earthquake that occurred prior to the dam's construction.

3. Feather River Basin Hydroclimatic Characteristics

3.1 Temperature

The Feather River Basin as well as the city of Oroville are characterized by a Mediterranean climate. Using the Köppen Climate Classification system, the subtype assigned to the area is “Csa”, indicating a “dry-summer subtropical” climate. For perspective, the city of Athens, Greece falls under the exact same Köppen subtype.

The average annual temperature of Oroville is usually around 16.6°C (61.9°F). The warmest month on average is July, with a mean temperature of 26.2°C (79.2°F), whereas the coldest month is usually January, with an average temperature of 7.6°C (45.6°F). However, the highest ever recorded temperature, 46.7°C (116.0°F) occurred during June, and the lowest record was -10.6°C (13.0°F), documented during December.

Below, the temperature data from (Weatherbase, 2017) is compared to a daily average temperature time series from the weather station located near the Oroville Dam spillway. This data set is provided by the California Data Exchange Center of the DWR (California Department of Water Resources, 2017). From this time series, monthly and annual averages are calculated using MATLAB and displayed in the tables below. When calculating the time series, in order to ensure accurate results, only the years with 300 or more daily measurements are taken into account. This corresponds to one measurement per day for at least 10 months in an average year. In a similar vein, for the monthly analysis, only months with more than 25 days of measurements are taken into account. Thus, due to several gaps in the time series during July and August 2014, an average was not calculated for those months.

Table 2. Monthly temperature averages (°C) for the years 2006 to 2016.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2006	8.87	9.92	7.65	12.67	19.78	24.48	27.99	24.87	22.91	17.13	11.61	8.64
2007	7.80	9.13	13.98	14.89	19.62	23.19	25.86	25.70	20.70	15.63	13.26	7.24
2008	6.33	8.72	11.90	14.30	19.84	24.07	26.08	26.63	23.74	18.12	12.67	5.86
2009	9.03	8.41	10.99	14.31	20.34	21.98	26.43	25.45	24.26	16.06	11.57	6.47
2010	7.24	9.50	10.70	11.52	15.25	22.39	25.72	24.23	22.96	17.58	10.89	8.69
2011	6.95	8.19	9.53	12.74	14.89	20.28	24.28	25.11	24.76	17.51	10.72	8.98
2012	8.92	9.64	9.91	13.76	19.25	29.37	25.22	26.97	24.43	17.90	12.19	7.54
2013	7.03	9.29	12.89	16.63	19.77	23.50	27.19	24.61	21.63	17.49	12.94	7.85
2014	12.58	10.20	13.26	16.04	20.56	24.69	-	-	23.44	19.09	12.15	9.57
2015	9.46	11.79	15.09	16.39	18.71	26.24	26.59	25.54	23.69	20.88	10.63	7.99
2016	9.03	12.47	12.58	16.85	19.73	24.54	26.61	26.02	23.41	16.63	12.61	8.15
Average	8.48	9.75	11.68	14.55	18.89	24.07	26.20	25.51	23.27	17.64	11.93	7.91

Table 3. Annual temperature averages in (ºC) for the years 2006 to 2016.

Year	Temperature (ºC)
2006	16.41
2007	16.46
2008	16.54
2009	16.32
2010	15.59
2011	15.37
2012	16.87
2013	16.79
2014	16.74
2015	17.78
2016	17.39
Average	16.57

The overall average annual temperature is 16.57ºC (61.83ºF), almost identical to the annual average given by (Weatherbase, 2017). Furthermore, when comparing the monthly averages to the overall mean value, it is evident that the hottest months in Oroville are the summer months, June, July, and August, followed by a cool period between October and March. The coldest months are December, January and February, with temperatures near half of the overall average. Finally, while there seems to be a warming trend for the last few years, the available record length is not enough to derive any substantial conclusions about climate change having affected the region.

3.2 Precipitation

According to (Weatherbase, 2017), precipitation in the Feather River basin occurs most usually during the cooler months, in rare yet intense events. On average, there are only 57 days of precipitation per year, and 35.9 of those are liquid. The driest month on average is July, with an average of 0 mm of precipitation. Conversely, the wettest month on average is January with an estimated 144.8 mm (5.7 inches). The total average precipitation for the year in Oroville is around 703.6 mm (27.7 inches).

Weatherbase's data is compared to daily precipitation data gathered near the Oroville Dam site (California Department of Water Resources, 2017). The available record starts at January 1st, 1987. Tables 4 and 5 contain monthly and annual precipitation data in millimeters, calculated from the original daily measurements. During the calculation of these time series, the same restrictions as those for the temperature analysis were applied, to ensure accuracy.

Table 4. Monthly total precipitation (mm) for the years 1987 to 2016 at Oroville Dam.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1987	-	465.33	325.12	-	-	-	173.74	1.02	1.02	-	-	-
1988	111.76	-	167.64	-	-	-	-	-	-	-	-	16.26
1989	109.73	0.00	58.928	286.51	26.42	11.18	18.29	0.00	2.54	71.12	87.88	42.67
1990	1.02	214.38	49.784	0.00	14.22	132.08	0.00	0.00	0.00	0.00	0.00	20.32
1991	49.78	34.54	89.408	401.32	19.30	28.45	15.24	0.00	0.00	0.00	7.11	21.34
1992	59.94	78.23	224.536	98.55	67.06	0.00	26.42	0.00	0.00	0.00	57.91	12.19
1993	199.14	310.90	248.92	108.71	59.94	53.85	35.56	0.00	20.32	0.00	27.43	62.99
1994	-	114.81	150.368	12.19	46.74	30.48	0.00	0.00	0.00	1.02	38.61	143.26
1995	19.30	456.18	24.384	390.14	67.06	82.30	60.96	0.00	0.00	0.00	0.00	3.05
1996	213.36	163.58	216.408	68.07	125.98	118.87	3.05	0.00	0.00	0.00	60.96	84.33
1997	329.18	334.26	6.096	47.75	22.35	14.22	19.30	0.00	16.26	10.16	64.01	167.64
1998	76.20	361.70	339.344	143.26	82.30	124.97	18.29	0.00	0.00	6.10	36.58	211.33
1999	55.88	117.86	230.632	54.86	44.70	4.06	4.06	0.00	0.00	0.00	51.82	106.68
2000	7.11	194.06	354.584	102.62	52.83	32.51	8.13	0.00	0.00	13.21	106.68	24.38
2001	30.48	130.05	197.104	71.12	46.74	0.00	4.06	0.00	0.00	10.16	33.53	179.83
2002	246.89	106.68	46.736	103.63	16.76	28.45	0.00	0.00	0.00	0.00	0.00	79.25
2003	368.81	121.92	72.136	67.06	188.98	39.62	0.00	0.00	26.42	0.00	0.00	97.54
2004	261.11	106.68	227.584	44.70	8.13	3.05	0.00	0.00	0.00	4.06	84.33	62.99
2005	244.86	120.90	66.04	119.89	47.75	100.58	36.58	0.00	0.00	1.02	48.77	106.68
2006	336.30	111.76	108.712	268.22	214.38	10.16	0.00	0.00	0.00	0.00	4.06	86.36
2007	167.64	2.03	224.536	13.21	65.02	20.32	13.21	9.14	0.00	12.19	49.78	31.50
2008	109.73	198.12	88.392	9.14	18.29	1.02	0.00	1.02	0.00	0.00	53.85	62.99
2009	101.60	68.07	230.632	72.14	15.24	77.22	10.16	0.00	0.00	6.10	43.69	51.82
2010	103.63	222.50	102.616	66.04	141.22	30.48	1.02	0.00	0.00	0.00	102.62	102.62
2011	230.63	51.82	138.176	237.74	25.40	75.18	40.64	0.00	19.56	0.00	69.09	44.20
2012	5.59	135.13	50.8	226.57	93.47	0.00	5.08	0.00	0.00	0.00	44.70	147.32
2013	249.94	26.42	20.32	67.06	17.27	11.18	23.37	0.00	0.00	18.29	2.03	49.78
2014	10.16	15.24	212.344	166.62	12.19	10.16	0.00	0.00	11.18	22.35	57.91	91.44
2015	338.33	1.02	93.472	10.16	64.01	0.00	4.06	6.10	1.02	7.11	12.19	73.15
2016	140.21	245.87	32.512	234.70	23.37	10.16	7.11	0.00	0.00	0.00	133.10	77.22
Average	149.23	155.52	146.61	124.71	58.11	37.52	18.22	0.60	3.39	6.53	45.67	77.97

Table 5. Annual total precipitation for the years 1987 to 2016 at Oroville Dam.

Year	Total Precipitation (mm)	Total Precipitation (inches)
1987	1077.98	42.44
1988	293.62	11.56
1989	606.55	23.88
1990	480.57	18.92
1991	676.66	26.64
1992	764.03	30.08
1993	1043.43	41.08
1994	441.96	17.4
1995	1297.43	51.08
1996	1170.43	46.08
1997	778.26	30.64
1998	1379.73	54.32
1999	621.79	24.48
2000	919.48	36.2
2001	919.48	36.2
2002	750.32	29.54
2003	874.78	34.44
2004	786.38	30.96
2005	984.50	38.76
2006	971.30	38.24
2007	550.67	21.68
2008	534.42	21.04
2009	678.69	26.72
2010	999.74	39.36
2011	707.39	27.85
2012	953.01	37.52
2013	245.87	9.68
2014	937.77	36.92
2015	412.50	16.24
2016	979.42	38.56
Average	794.61	31.28
St. Deviation	278.98	10.98
Skewness Coefficient	-0.02	-0.02
Excess Kurtosis Coefficient	-0.33	-0.33

From the analysis of Oroville Dam’s monthly precipitation data, it becomes clear that the average hydrologic year follows a pattern of extremely dry summers with almost no precipitation at all, followed by very wet winters where most of the total annual precipitation is produced. Specifically, the driest months are August, September, and October, whereas the wet period starts in November and lasts until April. These findings agree with the US Army Corps of Engineers analysis of the Feather Basin's climate prior to the dam's construction (US Army Corps of Engineers, 1970). The lowest average precipitation occurs during August (0.60 mm – 0.02 inches), and the maximum average value occurs during February (155.52 mm – 6.12 inches). In addition, the annual total precipitations seem to fluctuate significantly from year to year, yet the overall average, 794.61 mm (31.28 inches) is reasonable given the area's climate. Figure 2 below contains a plot of the data from Table 5, compared to the overall average value.

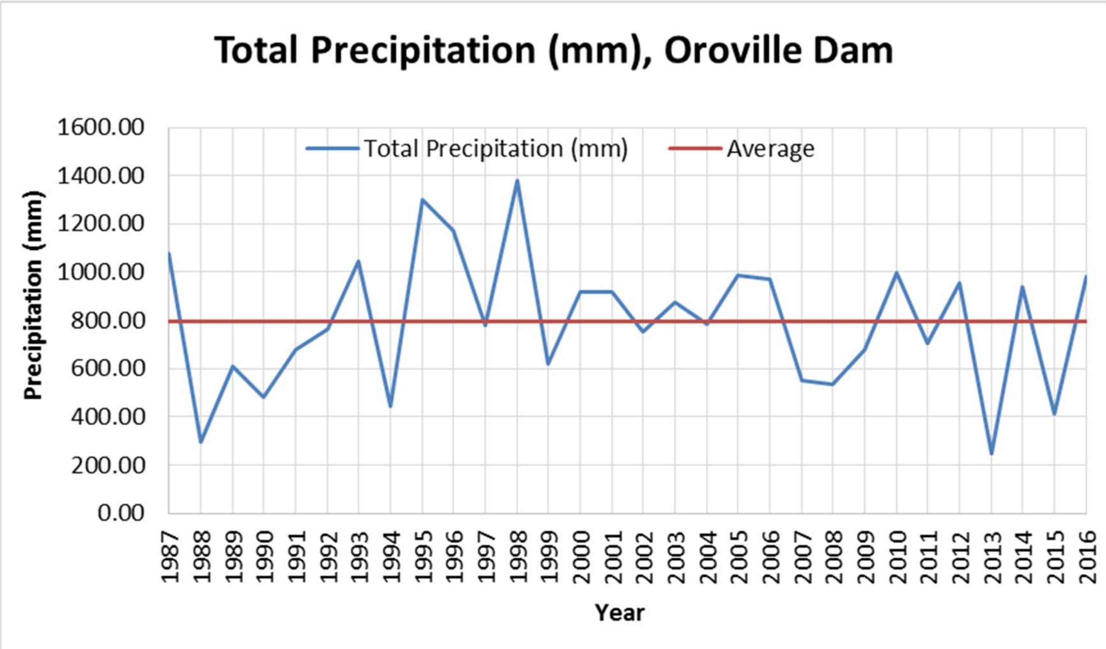
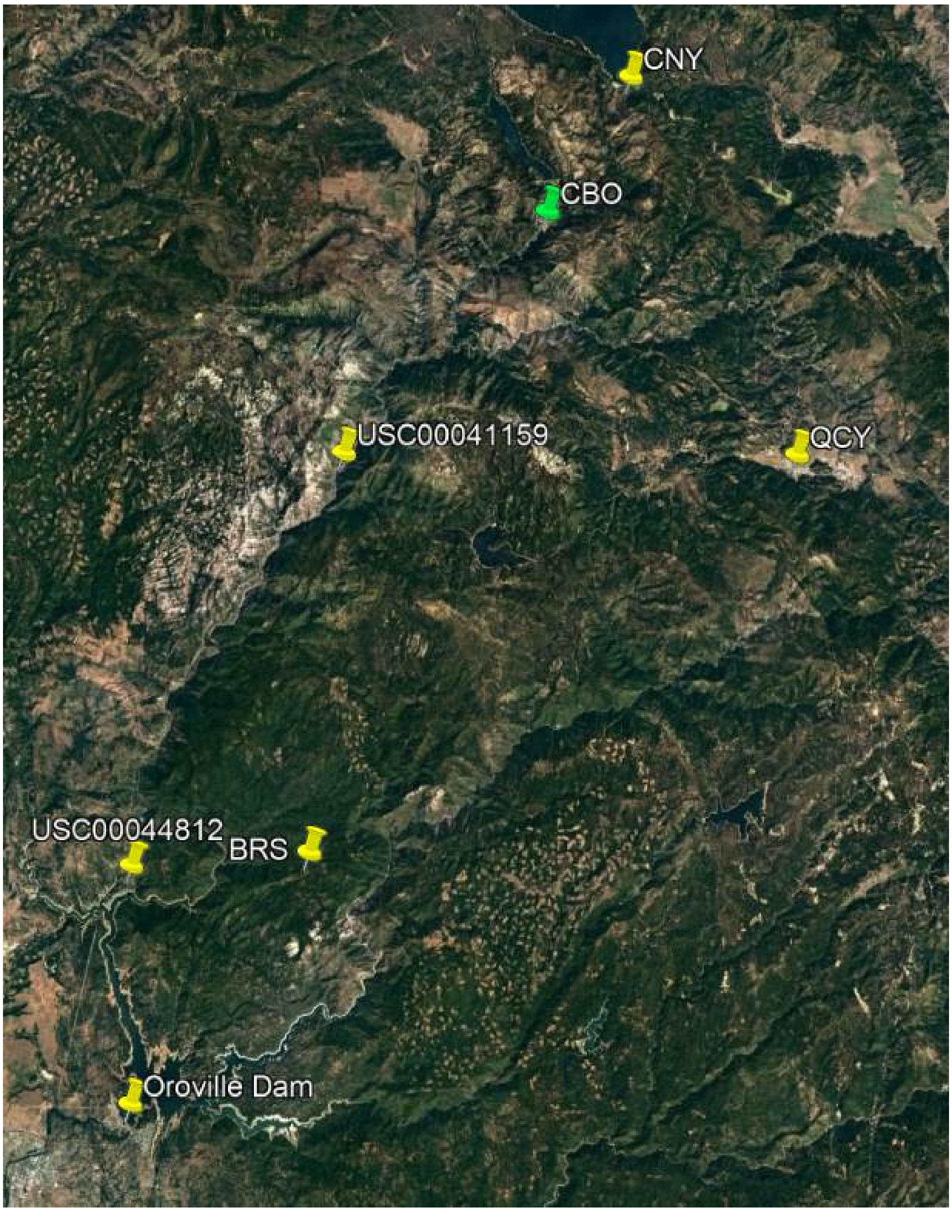


Figure 2. Annual total precipitation at Oroville Dam for the years 1987 to 2016, compared to the overall average.

However, due to the extent of the Feather River basin, more precipitation data is required in order to properly examine the area's hydrology. The National Oceanic and Atmospheric Association (Menne, et al., 2015) contains a database of several precipitation stations around the world, including two in the vicinity of Lake Oroville. Moreover, monthly total precipitation data was available through the California Data Exchange Center website from stations operated either by the DWR or by Pacific Gas & Electric (California Department of Water Resources, 2017). The analyzed station names and other pertinent data are given below, as well as a map of their locations.

Table 6. Analyzed precipitation measurement stations and related information.

Station Name	Station ID	Latitude	Longitude	Data Source	Record
Las Plumas	USC00044812	39.6833	-121.4833	NOAA	1913-1967
Bucks Creek	USC00041159	39.9372	-121.314	NOAA	1959-2016
Quincy	QCY	39.935	-120.95	CA DWR/O & M	1905-1979
Canyon Dam	CNY	40.167	-121.083	Pacific Gas & Electric	1907-1982
Caribou	CBO	40.085	-121.15	Pacific Gas & Electric	1920-1995
Brush Creek	BRS	39.692	-121.339	CA DWR/O & M	1935-2010



Map 3. Analyzed precipitation measurement stations and related information. Source: Google Earth (2017)

Using the same method and restrictions as before, total annual precipitation time series are created for each station. Complete tables can be found in Appendix A, whereas Table 7 contains a statistical summary of the results.

Table 7. Overall averages and standard deviations of annual total precipitation for each station.

Station ID	Average (mm)	St.Deviation (mm)	Average (inches)	St.Deviation (inches)
USC00044812	1173.4	384.7	46.19	15.14
USC00041159	1590.6	653.1	62.62	25.71
QCY	1001.97	326.17	39.44	12.84
CNY	930.93	257.04	36.65	10.12
CBO	1014.53	300.56	39.94	11.83
BRS	1803.26	575.6	70.99	22.66

Analysis of the precipitation measurement stations from NOAA's database returns results very similar to that of the station near Oroville Dam. However, the average precipitation and standard deviations are much higher, indicating a pattern in the behavior in the Feather Basin's climate, where extremely dry years are followed by intense precipitation events. In order to determine whether this pattern is uniform across the entire Feather River Basin, correlations between the annual total precipitations are calculated, shown below. For the correlation analysis, only common years between stations are taken into account.

Table 8. Correlation of total annual precipitation between stations.

Station ID	USC00044812	USC00041159	QCY	CNY	CBO	BRS
USC00044812	1.00	0.68	0.81	0.87	0.90	0.88
USC00041159		1.00	0.75	0.75	0.89	0.68
QCY			1.00	0.75	0.88	0.84
CNY				1.00	0.87	0.78
CBO					1.00	0.91
BRS						1.00

As is evident from this analysis, the precipitation stations have a high correlation on an annual basis. Only station USC00041159 has moderate-to-high calculated coefficients, and that is likely due to the relatively low amount of common years of recorded data compared to the other stations.

The fact that the basis of the analysis is annual means that any possible day-to-day differences in precipitation data are eliminated. While this is useful for determining the overall climate of the Feather Basin, a daily or even hourly basis of measurements would be required in order to properly capture how storms vary over the area and thus create an accurate simulation of weather events. A study of precipitation-runoff processes in the Feather River basin (Koczot, Jeton, McGurk, & Dettinger, 2005) indicates that while there are significant differences between calculated daily precipitation data from station to station, on a monthly basis cross correlation coefficients between average values remain as high as 0.90 in the winter, dropping to 0.80 in the warmer period.

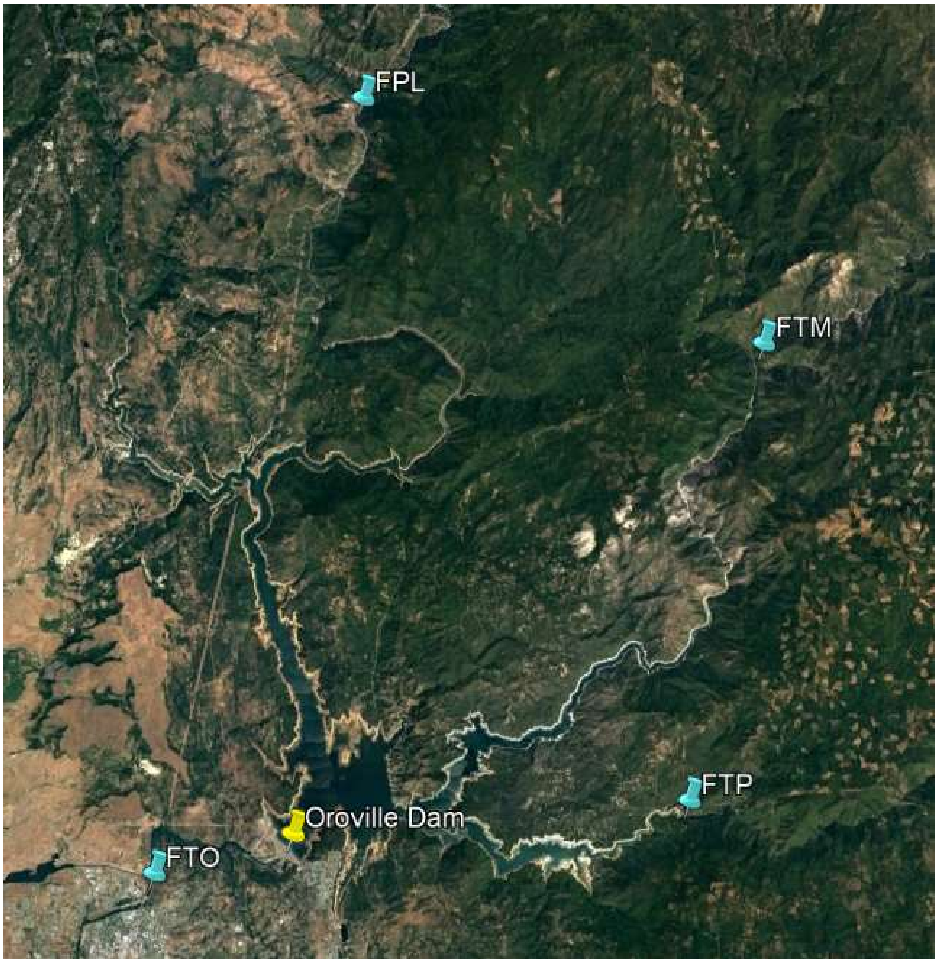
3.3 Streamflow

Precipitation in the Feather River Basin usually falls as snow during the winter at elevations above 5,000 feet and as rain at lower elevations, yet major warm storms may cause rain throughout the entire area regardless of altitude. Runoff of the Feather River is produced mainly from warmer-than-freezing temperatures during intense precipitation events in the winter. These events cause higher flows as a result of rain and melting snow from higher elevations. The highest flows occur from December through June, with April and May producing the most sustained amounts. However, more precipitation falls as rain in the Feather River area compared to nearby basins in the Sierra Nevada, resulting in lower overall streamflow peaks in April caused by snowmelt in the spring.

Full natural monthly flow data was gathered from (California Department of Water Resources, 2017), for four stations around Oroville Dam. Pertinent data can be found below, as well as a map of the station locations.

Table 9. Analyzed full natural monthly flow measurement stations and related information.

Station Name	Station ID	Latitude	Longitude	Record (Water Years)
FEATHER RIVER AT OROVILLE	FTO	39.522	-121.547	1905-06 to 2016-17
FEATHER MF NR MERRIMAC	FTM	39.708	-121.269	1907-07 to 1969-70
FEATHER NF AT PULGA	FPL	39.794	-121.451	1911-12 to 1993-94
FEATHER SF AT PONDEROSA	FTP	39.548	-121.303	1900-11 to 1991-92



Map 4. Map of analyzed full natural flow measurement stations in the Feather River basin. Source: Google Earth (2017).

Using the free HYDROGNOMON software tool (http://hydrognomon.org/), the monthly data are aggregated into annual time series. The basis of the analysis is in water years (October 1 to September 30), which is more appropriate for hydrology studies and also permits the inclusion of data from the latest completed water year, 2016-17. These analyzed annual flows are plotted below, and detailed tabular output can be found in Appendix A.

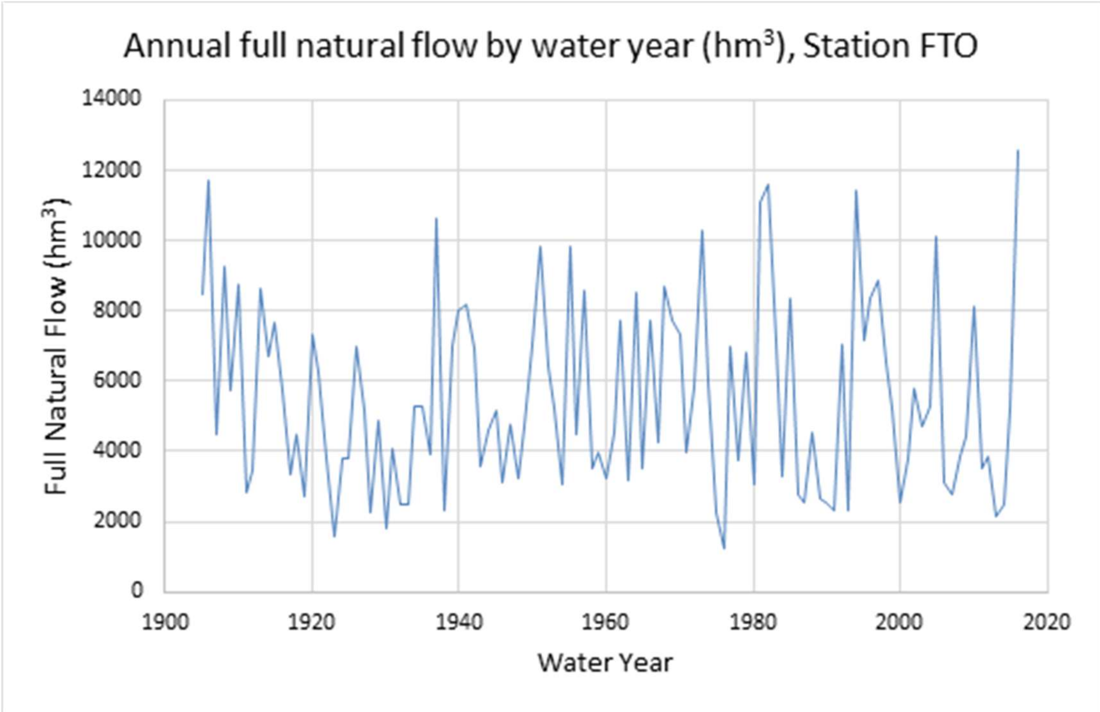


Figure 3. Annual full natural flow by water year, Feather River at Oroville (FTO) station.

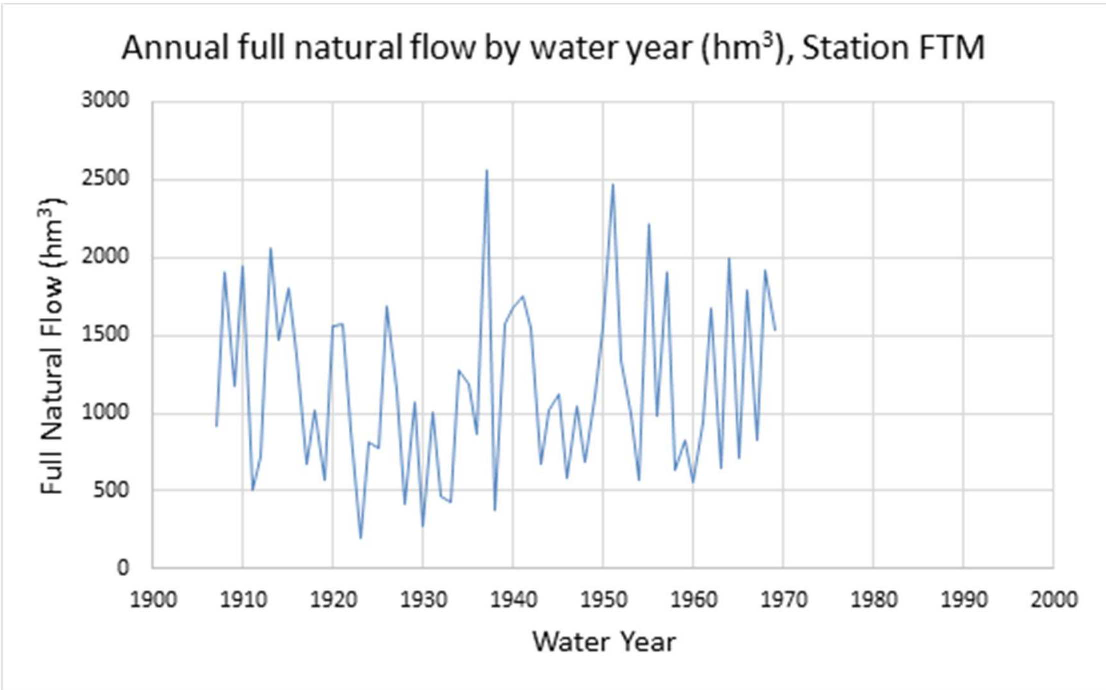


Figure 4. Annual full natural flow by water year, Feather Middle Fork near Merrimac (FTM) station.

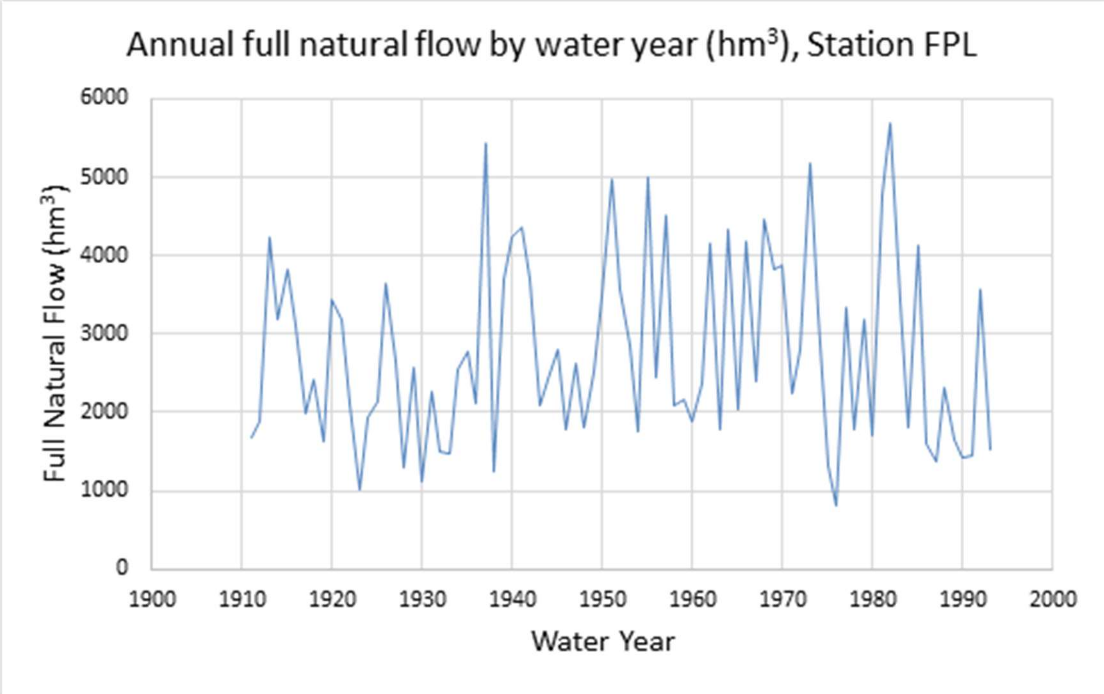


Figure 5. Annual full natural flow by water year, Feather North Fork at Pulga (FPL) station.

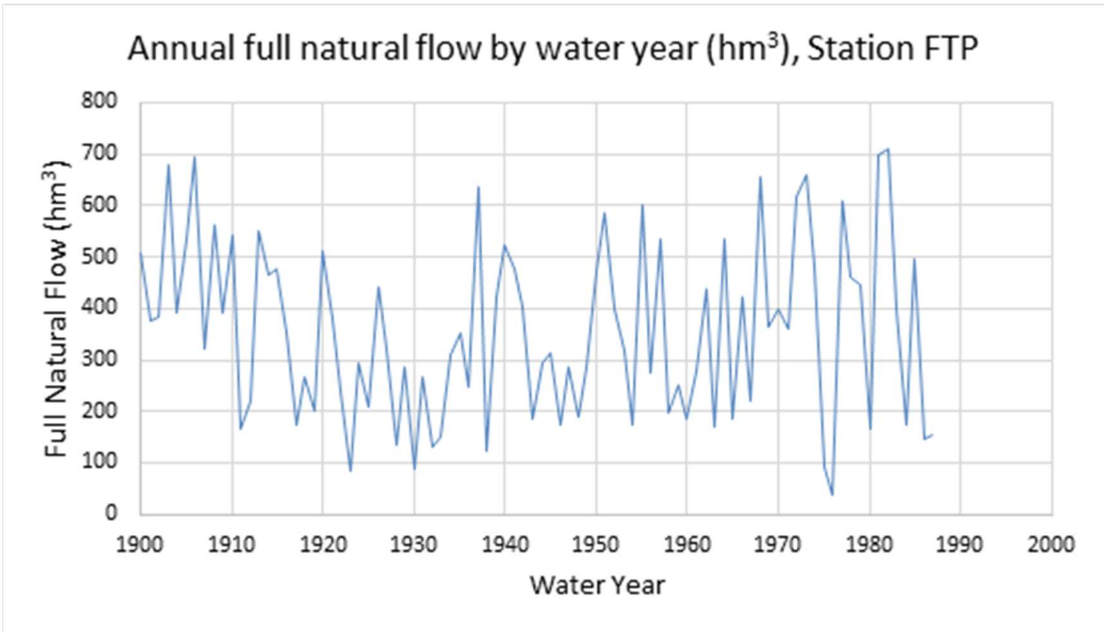


Figure 6. Annual full natural flow by water year, Feather South Fork at Ponderosa (FTP) station.

In addition, Table 10 below displays the average runoff per month compared to the annual average, in order to display the distribution of streamflow in an average water year.

Table 10. Average full natural monthly flow in hm3 compared to the annual average for each station.

Average Monthly Natural Flow								
Station	**FTO**		**FTM**		**FPL**		**FTP**	
	Flow (hm^3)	**% of Annual**	**Flow (hm^3)**	**% of Annual**	**Flow (hm^3)**	**% of Annual**	**Flow (hm^3)**	**% of Annual**
October	127.59	2.34%	18.71	1.63%	91.84	3.60%	6.45	1.87%
November	220.42	4.04%	34.56	3.00%	128.6	5.04%	14.98	4.33%
December	446.27	8.18%	84.95	7.38%	211.84	8.30%	28.93	8.37%
January	621.42	11.40%	116.42	10.12%	263.52	10.32%	40.62	11.75%
February	693.5	12.72%	139.36	12.11%	305.41	11.96%	50.57	14.62%
March	851.67	15.62%	171.77	14.93%	391.89	15.35%	56.14	16.23%
April	885.22	16.23%	240.32	20.89%	436.58	17.10%	58.23	16.84%
May	805.99	14.78%	214.45	18.64%	435.81	17.07%	53.31	15.42%
June	421.08	7.72%	92.62	8.05%	238.93	9.36%	22.62	6.54%
July	193.63	3.55%	28.75	2.50%	126.27	4.95%	8.54	2.47%
August	126.7	2.32%	14.49	1.26%	90.94	3.56%	4.75	1.37%
September	107.74	1.98%	12.32	1.07%	75.3	2.95%	4.54	1.31%
Annual	**5452.56**	**100.00%**	**1150.45**	**100.00%**	**2552.91**	**100.00%**	**345.82**	**100.00%**

Especially for the Feather River at Oroville (FTO) station, the California Department of Water Resources creates the monthly flow database using a flow reconstruction procedure that takes the following factors into account (California Department of Water Resources); (Koczot, Jeton, McGurk, & Dettinger, 2005):

Table 11. Components of full natural monthly flow reconstruction for the Feather River at Oroville (FTO) station, as given by the DWR. Sources: DWR, (Kathryn M. Koczot, 2005)

(1) + Measured streamflow at USGS 11407000.
(2) + Thermalito Afterbay releases to the Feather River, through the Thermalito Afterbay River Outlet
(3) + Diversions at the Thermalito Complex
(4) + Thermalito Irrigation District and Butte County diversions (California Water Service) from the Thermalito Power Canal Diversion
(5) + Gain in storage of Thermalito Complex (Diversion Pool, Forebay and Afterbay)
(6) + Evaporation at Thermalito Afterbay, Thermalito Forebay, and Diversion Pool
(7) + Lake Oroville gain in storage
(8) + Lake Oroville evaporation loss only. Zero when raining
(9) + Palermo diversion (from Lake Oroville) and Bangor Canal diversion
(10) + Oroville-Wyandotte Canal, also known as Forbestown Ditch (from South Fork), and Hendricks and Miocene Canals (from West Branch)
(11) + Storage gain at Lake Almanor, Mt. Meadows, Butt Valley, Bucks Lake, Frenchman, Antelope, Lake Davis, Little Grass Valley, and Sly Creek reservoirs
(12) + Estimated evaporation for reservoirs above the station, computed as 1.4 times the Lake Almanor evaporation, based on a monthly capacity. The evaporation table is from the Great Western Power Company (PG&E predecessor)
(13) − Slate Creek Tunnel import from the Yuba River basin, which flows into the South Fork at the Sly Creek Reservoir
(14) − Little Truckee River import into Sierra Valley
(15) + Depletion for upstream irrigation and consumptive use

The gaging station upon which all other calculations are based is USGS gaging station 11407000, which measures discharge from Lake Oroville in cubic feet per second (cfs) since 1901. These measurements are then reconstructed with corrections due to factors such as evapotranspiration and streamflow regulations from numerous other smaller basins in the Feather River area, as described above.

The US Geological Survey National Water Information System (USGS, 2017) contains daily discharge data for station 11407000. Using HYDROGNOMON, this data was converted into cubic hectometers and then aggregated into a monthly time series, which is plotted below.

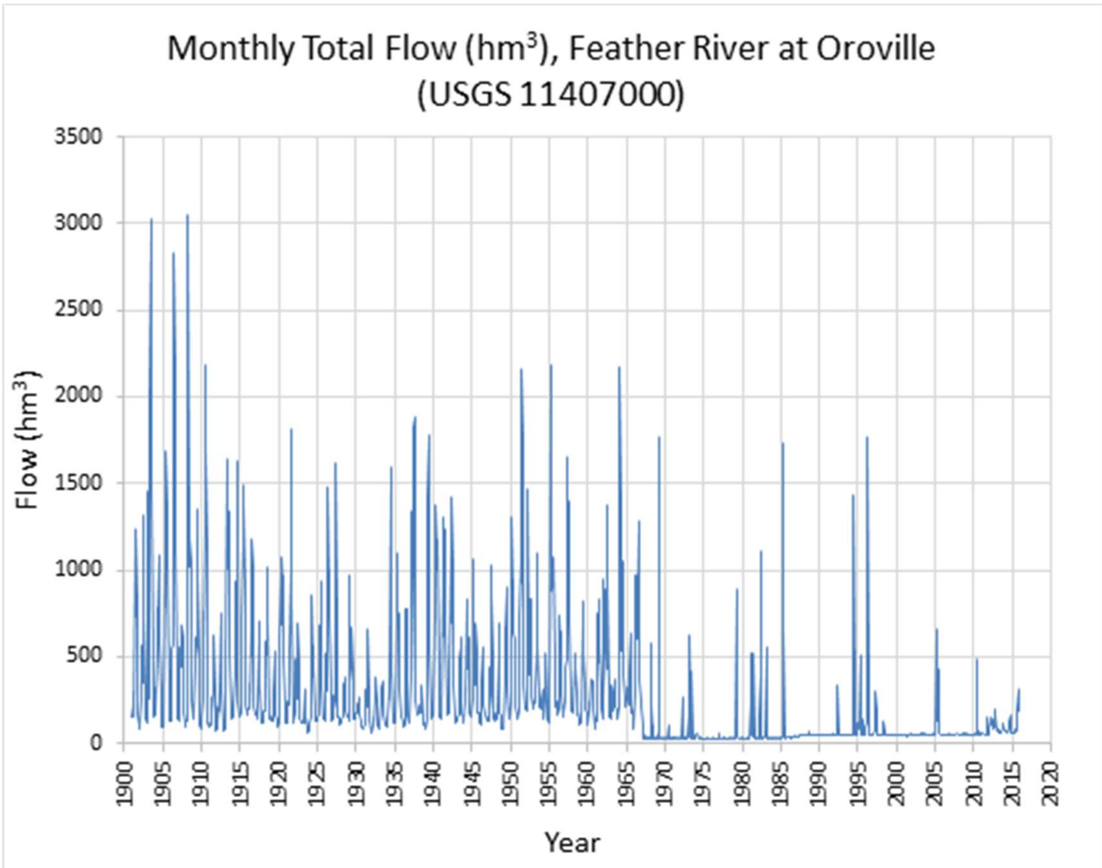


Figure 7. Monthly total flow in hm3, Feather River at Oroville, station USGS 11407000.

As seen in the graph above, after the year 1967, minimum monthly flows decrease significantly, down to almost zero. This is due to the fact that Oroville Dam was completed around that time, thus heavily regulating Feather River flows from that point onwards. In addition, after the construction of Oroville Dam, the gage was moved further downstream, which may also have an impact on measurements after the station's relocation (Markham, Anderson, Rockwell, & Friebel, 1996).

The monthly results of this station have an extremely high correlation (0.98) with the monthly flow data up to water year 1967-68. However, correlation between all of the available monthly data returned a coefficient of 0.75. While still high, it does indicate that the flow regulation from Oroville Dam and the relocation of the USGS station influenced the latter's measurements from then on.

By aggregating the measurements from USGS 11407000 even further to an annual water year basis, it is possible to directly compare the measurements from the CDEC FTO station to the USGS results, as displayed in the graph below.

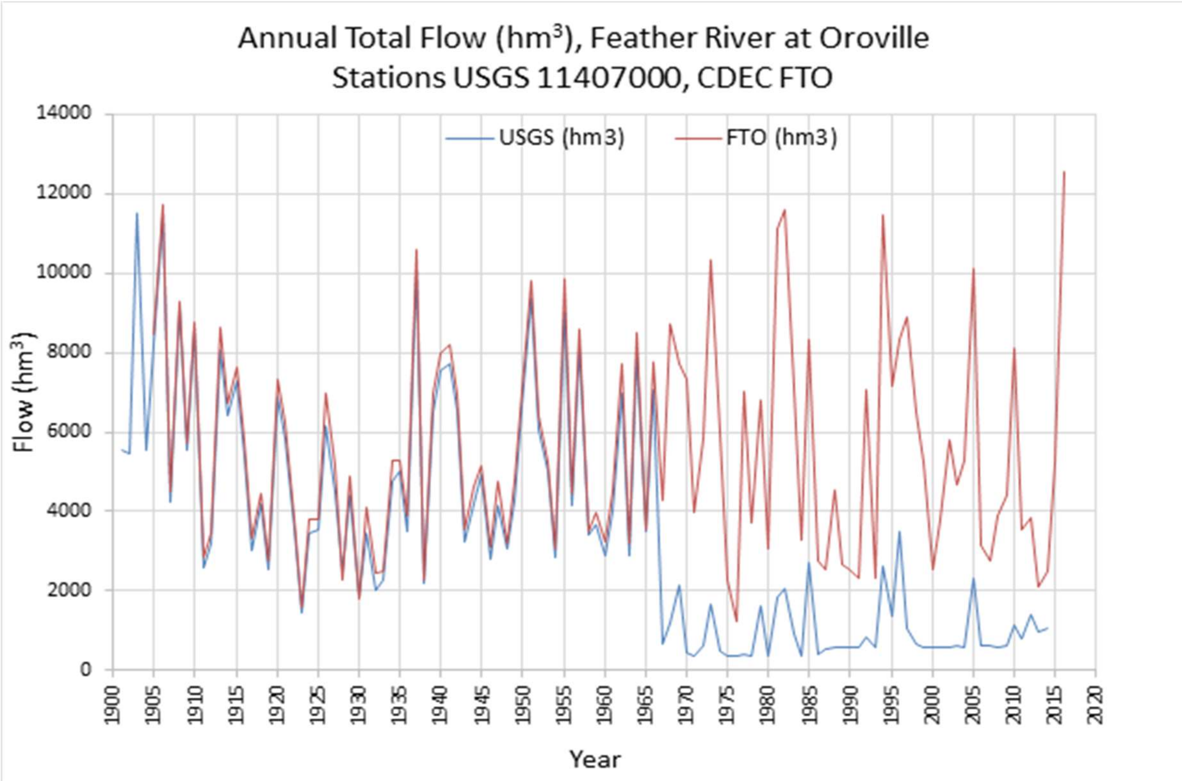


Figure 8. Annual total flows in hm3, Feather River at Oroville, stations USGS 11407000 and FTO.

By directly comparing the streamflow measurements from both stations, it becomes clear that the DWR's flow reconstruction process for station FTO as described above most likely began after Oroville Dam's completion, and attempts to simulate runoff characteristics without the regulatory effects from the various reservoirs in the Feather River Basin. It is important to note that the complexity of the flow reconstruction method impacts the overall accuracy of the calculations. The US Geological Survey rates the accuracy of station 11407000 according to the stability and accuracy of stage and discharge measurements, as well as the interpretation of records (Bostic, Kane, Kipfer, & Johnson, 1996). Accordingly, PG&E rates the accuracy of its own flow reconstruction from the North and South Fork drainages at around 15 percent. Finally, the relocation of station 11407000 as well as assumptions regarding evapotranspiration and water demand for consumption and irrigation purposes could significantly impact the accuracy of the flow reconstruction for the DWR station FTO. While the US Geological Survey has not quantified the accuracy of the flow reconstruction results, the DWR assumes them to be within 5 to 10 percent of their true values. (DWR, 2001); (Koczot, Jeton, McGurk, & Dettinger, 2005)

3.4 Floods

Large floods in the Feather River basin occur due to severe winter rain storms, in some cases augmented by snowmelt. A typical event may last several days, but is actually not a single storm, but a short sequence of smaller individual storms in quick succession. In these cases, runoff can combine to produce high-peak intense flows downstream with a variety of flood characteristics. However, while streamflow accumulates quickly in the upper tributaries of the Feather River, the floods produced usually have a high peak but a short overall duration. Further downstream, the slopes of the riverbanks are less steep, resulting in more prolonged inundation in the river's lower reaches. Additionally, ever since the construction of various protective reservoirs, dams and levees in the Feather Basin, a significant flood could also be a result of a failure or overtopping of any one of these structures.

There are two types of floods in the basin above Oroville Dam: Rain floods, with short durations and high peaks, and floods occurring due to snowmelt, which result in sustained high flows over a period of up to several weeks. The regulatory nature of several smaller dams and diversions upstream helps delay the effects of peak flood events from rapidly affecting downstream areas. Furthermore, due to the canyon-like nature of the Feather River's upper reaches, even in more severe flood events streamflow is confined within natural stream channels and rarely causes damage upstream of Oroville Dam. (US Army Corps of Engineers, 1970)

Prior to the dam's completion, several floods had impacted the surrounding area, with the largest recorded flow until that point having occurred in 1964, during Oroville Dam's construction. A report by (Lamontagne, et al., 2012) contains unregulated, annual maximum flow data for the Feather River at Oroville station resulting from rainfall for 1-day, 3-day, 7-day, 15-day, and 30-day durations as provided by the US Army Corps of Engineers. Detailed tabular output can be found in Appendix A, and the most significant events are described below. It is important to note certain constraints and assumptions related to this data set. First off, only floods resulting from n-day rainfall data are measured, not floods due to snowmelt. However, due to the nature of rainfall floods producing larger peaks overall, the results are most likely suitably accurate. Secondly, maximum flows are calculated as the n-day average of the highest flows in each water year. Each n-day period is useful for different aspects of reservoir management. For example, the 3-day duration provides the most critical flood flow estimate most necessary for dam operation, whereas a 7-day scale event could be the result of two back-to-back 3 day storms, which is a likely event in the

Feather River Basin (Hickey, et al., 2002); (Cudworth, 1989); (US Army Corps of Engineers, 1970). The most intense floods from this analysis can be found below.

Table 12. Historical floods, Feather River at Oroville. Sources: USACE, (Lamontagne, 2012)

	1-day		3-day	
Water Year	**Date**	**Flow (m^3/s)**	**Date**	**Flow (m^3/s)**
1903-04	24-Feb	3001.59	18-Mar	2501.23
1906-07	19-Mar	5295.25	18-Mar	4256.87
1908-09	16-Jan	3879.41	14-Jan	3643.53
1927-28	26-Mar	3544.42	25-Mar	3139.77
1937-38	11-Dec	4501.81	10-Dec	3004.98
1939-40	30-Mar	3815.98	27-Feb	3055.67
1955-56	23-Dec	5140.36	22-Dec	4160.03
1964-65	23-Dec	5055.97	22-Dec	4683.32
1979-80	13-Jan	3896.96	12-Jan	3032.73
1985-86	17-Feb	6145.32	17-Feb	5295.53
1994-95	10-Mar	3799.78	9-Mar	3221.98
1996-97	1-Jan	8860.14	31-Dec	6923.04

From the floods mentioned above, (US Army Corps of Engineers, 1970) contains further information for some of those occurring prior to Oroville Dam's construction. For example, the flood of March 1907 occurred due to a combination of heavy rainfall mixed with melting snow due to unusually high temperatures for the season. Peak flow at Oroville reached 6,513 m^3/s (230,000 cfs).

Image 1. A crowd on Myers Street, Oroville, California after the flood of March 1907.

Next, the flood of December 1955 was caused by similar conditions, peaking at 5,750 m^3/s (203,000 cfs). As mentioned earlier, it was the severe property damages and deaths caused by this flood that prompted the State to present flood control countermeasures in the Feather River Basin even before the funding for the Oroville complex was finally approved. Finally, the greatest flood to occur prior to the dam's completion began in December 1964, following a typical storm pattern for the area, but with significantly higher duration, resulting in nearly 60 days of heavy precipitation over the basin. This storm came in four distinct waves, with the peak occurring during the 23rd of December, where about 330 mm (13 inches) of rain fell. This event also melted snow from previous events resulting in a flood peak of 7,080 m^3/s (250,000 cfs). However, the only partially built Oroville Dam was even then capable of significantly halting the incoming flow, down to a maximum of 4,474 m^3/s (158,000 cfs).

4. Oroville Dam Design Elements

4.1 Dam Configuration

Oroville Dam is a zoned earth-fill embankment structure with a maximum height of 235 m (770ft.) above streambed excavation. The embankment itself has a volume of approximately 61 million m^3 (80 million cubic yards) and is comprised of an inclined impervious core atop a concrete foundation, supplemented by zoned earth-fill sections on both sides.

Image 2. Aerial view of Oroville Dam. Source: California Department of Water Resources (2008).

Oroville Dam's spillway, located on the right abutment of the main dam, is comprised of two independent elements: a gated flood control outlet and an uncontrolled emergency spillway. The former consists of an unlined approach channel, a gated headworks, and a lined chute approximately 930 m (3050 ft.) in length, extending down to the Feather River. The latter is an ungated concrete ogee weir with the crest set at elevation 274.62 m (901 ft.), just one foot above maximum storage level (elevation 274.32 m or 900 ft.). The area below the emergency spillway is not lined with concrete, meaning that when it is put to use, flow will spill over natural terrain.

Most of the streamflow released from Lake Oroville passes through the Edward Hyatt Powerplant, located in the dam's left abutment. Total output of the plant is estimated at 678.75 MW, produced by 6 Francis-type turbines, rated at approximately 115 MW each. It is also capable of pump-storage, which offers the potential to maximize the value of generated energy. This station is underground, with dimensions of approximately 168 m by 21 m by 43 m (550 feet long, 69 feet wide, 140 feet high).

The intake for the Hyatt Powerplant is a sloping concrete structure, built just upstream of the Oroville Dam's left abutment. It is comprised of two parallel channels, one for each of the two 6.7 m (22 ft.) diameter penstock tunnels. The openings of the intake are protected from incoming debris by steel trashracks.

Beneath the trashracks, a square shutter system 12.2 m (40 ft.) wide determines the level and temperature of the water withdrawn from the reservoir. Especially the temperature of the water withdrawn can be critical for local agricultural purposes, as well as for the local wildlife. In addition, in case of an emergency, the penstocks can be closed through hydraulically activated gates located at the base of the intake channels. However, under standard operation conditions, any discharges from the Hyatt Powerplant are conveyed to the Feather River with the use of the dam's two former diversion tunnels, each 10.7 m (35 ft.) in diameter. In the event of a prolonged outage at the plant, water flows directly through these into a downstream river outlet, with a maximum release of 151.2 m^3/s (5,400 cfs). Pertinent data related to Oroville Dam and related facilities can be found below. (California Deparment of Water Resources, 1974) (California Department of Water Resources, 2017).

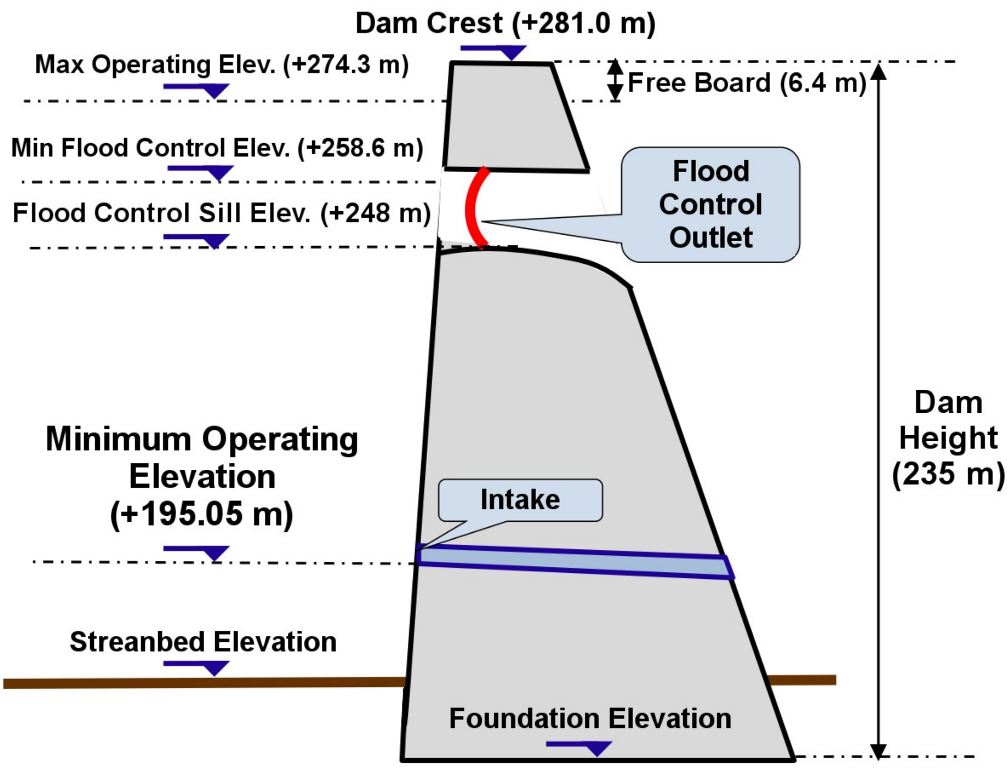


Figure 9. A typical section of Oroville Dam, with important elevation data. Not to scale. Adapted from (Efstratiadis, Michas, & Dermatas, 2017)

Table 13. Statistical summary of Oroville Dam and related facilities.

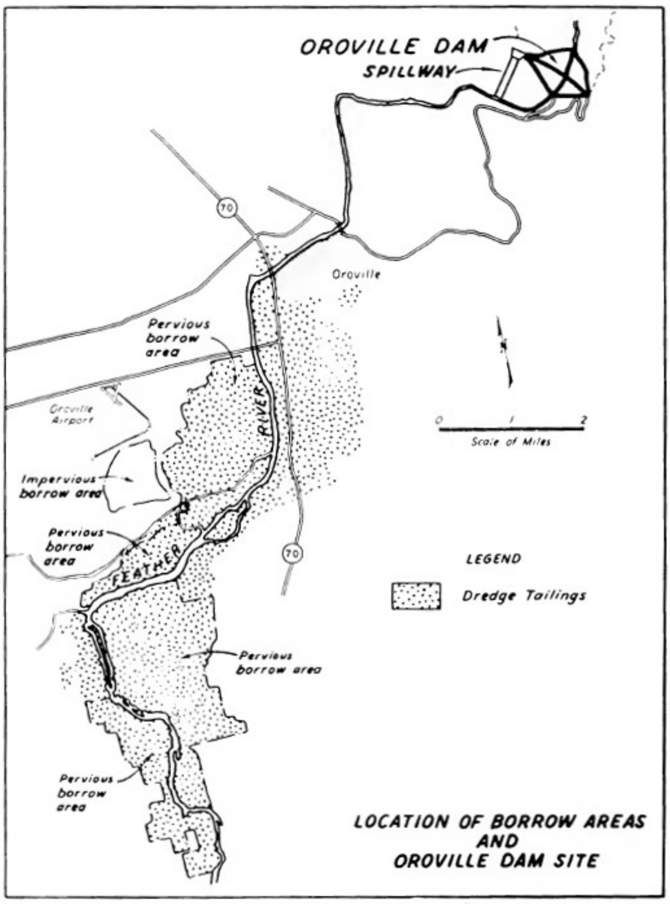
Pertinent Data				
Oroville Dam				
Crest Elevation	922	ft	281.03	m
Height	742	ft	226.16	m
Total Freeboard	108.4	ft	33.04	m
Operating Freeboard	21	ft	6.40	m
Streambed Elevation	180	ft	54.86	m
Lake Oroville				
Maximum Operating Storage	3537577	AF	4364	hm^3
Storage, Flood Control Pool	2778000	AF	3427	hm^3
Dead Pool Storage	29638	AF	37	hm^3
Max Operating Surface Elevation	900	ft	274.32	m
Surface Elevation, Flood Control	848.5	ft	258.62	m
Min Operating Surface Elevation	640	ft	195.05	m
Dead Pool Surface Elevation	340	ft	103.63	m
Max Operating Surface Area	15805	acres	63.96	km^2
Min Operating Surface Area	5838	acres	23.63	km^2
Reservoir Area	15805	acres	63.96	km^2
Drainage Area	3607	sq miles	9342.09	km^2
Spillways				
Emergency Spillway Crest Elevation	901	ft	274.62	m
Emergency Spillway Design Capacity	350000	cfs	9910.90	m^3/s
Main Spillway Flood Control Sill Elev	813.6	ft	247.99	m
Main Spillway Design Capacity	277000	cfs	7843.77	m^3/s
PMF 1968 - Combined Inflow	720000	cfs	20388.13	m^3/s
PMF 1968 - Combined Outflow	624000	cfs	17669.71	m^3/s
Maximum Surface Elevation	917	ft	279.50	m
Powerplant Intake				
Maximum generating release	16900	cfs	478.55	m^3/s
Pumping Capacity	5610	cfs	158.86	m^3/s
Outlet Works				
River Outlet Capacity	5400	cfs	152.91	m^3/s
Palermo Outlet Tunnel Capacity	40	cfs	1.13	m^3/s

4.2 Purpose

Oroville Dam and its facilities provide a number of functionalities to its users, including water conservation for irrigation and general consumption purposes, power generation, and the Lake Oroville ecosystem, which is a center of recreational activities. The combined capacity of the Hyatt Powerplant and the downstream Thermalito Complex is 725 MW, resulting in an output of over 2 TWh per year. The approximately 64 square kilometers of Lake Oroville offer a variety of water-based recreational activities and receive a substantial amount of tourists all year round (California Deparment of Water Resources, 1974) (California Department of Water Resources, 2017).

4.3 Construction Materials

The ultimate choice of an embankment-type structure for Oroville Dam relied on the availability of suitable building materials. As mentioned earlier, extensive dredge tailing fields left over from the gold mining era provided an adequate supply of earth and rock for all the zones of an earth-fill dam.



Map 5. Location of borrow areas for the Oroville Dam embankments. Source: California Deparment of Water Resources (1974).

The impervious core borrow area was given the top priority, because its proximity to a pervious borrow area allowed the common transportation of both material types. Furthermore, a vertical excavation type was selected, at a depth that would produce the desired gradation for the material.

Exploration of the dredger tailings was carried out in two phases. During the first phase, Oroville Dam was still being proposed as a concrete structure, so material from the borrow areas was initially meant to be used as aggregate. A problem that arose when attempting to analyze samples of the borrow area was the coarseness of the material and the fact that parts of it were below the static water level. This was eventually solved using a hole excavator, consisting of a carrier beam with a mounted clamshell bucket, operated from the back of a truck using a hydraulically controlled winch. The greatest possible excavation depth that could be achieved with this method was 37 feet, and was used to penetrate gravelly materials above the water level. Below it lied quantities of sand, which were excavated with a piston-operated cylindrical sucker. From 1956 to 1957, 70 holes on a 305 m (1000 ft.) grid spacing were drilled through the selected dredge tailing deposit area.

By 1959, Oroville Dam was determined to be built as a zoned embankment type, and the second phase of the borrow area exploration started. The purpose of this program was to examine the availability and suitability of the dredge tailings for the outer zones of the dam. First off, aerial photographs of the area were examined in order to pinpoint areas on the map that had similar ground characteristics and single them out for exploration. The fact that the necessary material was no longer concrete aggregate meant that extensive and detailed grading was not as important as before, and so the aforementioned hole excavator was not used further. Instead, the new selected methods were bulldozer trenches and dragline pits. The final program consisted of 71 dragline pits and 129 bulldozer trenches.

Pit exploration occurred as follows: First, the bulldozer leveled a site between two linear ridges and then pushing the material into the adjacent valleys. This was in order to produce an average gravel thickness, and also reduced the overall depth of excavation, making it easier to also obtain the required amount of sand from beneath the water table. After the bulldozer leveled the area, the dragline was moved in and test pits were excavated in order to collect a clean sand sample and accurately determine the sand table’s elevation. Afterwards, one large representative sample of about 8 m^3 was cut out of the pit wall with the dragline bucket. From this pile of gravel, smaller segments were taken to the laboratory for testing.

These bulldozer trenches were used to broadly classify the dredger tailings into clean, sandy, silty, and clayey gravels, and then depict them on a map and compute volumes for each type, all based on the thickness of the material during pit excavation. From this analysis, it was determined that there were approximately 107 million cubic meters (140 million cubic yards) of available coarse borrow material.

In the impervious borrow area, due its compactness and gravelly nature, it was necessary to use heavy-duty drill rigs with extra heavy kelleys, and a variety of bucket types (California Deparment of Water Resources, 1974).

4.4 Embankment Design

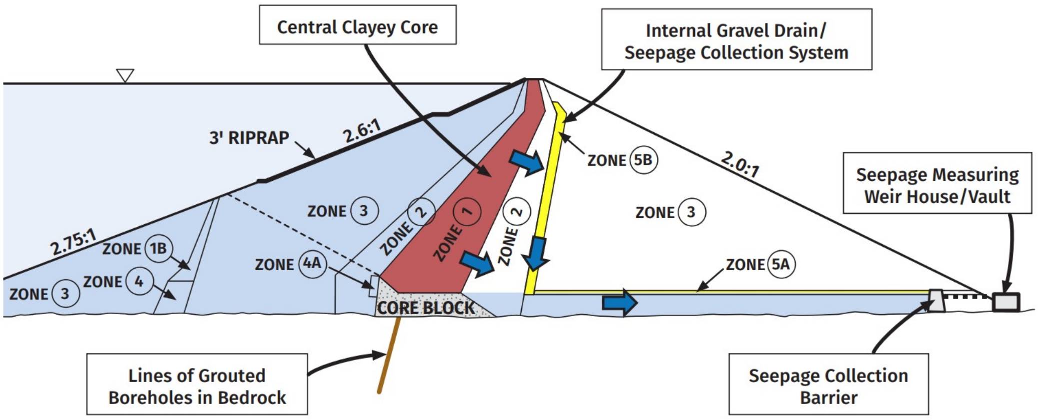


Figure 10. Cross section of Oroville Dam, including seepage barriers and the seepage collection system. Source: (California Department of Water Resources, 2017)

Oroville Dam's main embankment is comprised of the following zones (see figure above):

Zones 1, 1A, 1B: Impervious core from the deposit next to the pervious borrow areas. A well-graded mixture of silt, sand, gravels, and cobbles up to 180 mm in diameter. Compacted in 25.4 cm (10 inch) lifts by 100-ton pneumatic rollers.

Zone 2: Transition zone comprised of a well-graded mixture of silt, sand, gravels, cobbles, and boulders up to 380 mm in diameter. Compacted in 38 cm (15-inch) lifts by a smooth-drum vibratory roller.

Zone 3: Shell zone, comprised of mostly sands, gravels, cobbles, and boulders up to 610 mm in size. Compacted in 61 cm (24-inch) lifts by a smooth-drum vibratory roller.

Zone 4: Impervious core containing selected abutment stripping, between 15 and 45 percent passing standard No. 200 US Standard sieve with 200 mm maximum size. Compacted in 25.4 cm (10 inch) lifts by 100-ton pneumatic rollers.

Zones 5A, 5B: Drainage zones built out of gravels, cobbles, and boulders. Maximum of 12 percent larger than No.4 sieve size permitted. Compacted in 61 cm (24-inch) lifts by a smooth-drum vibratory roller.

The normal water surface was selected to be at 274.32 m (900 ft.) even at the time when Oroville Dam was still being considered at as a concrete-type structure. Factors deciding this were the dedicated reservoir volume requested during design, as well as the elevations of the spillway site and nearby Parish Camp Saddle Dam and Bidwell Canyon were at about the same level.

Whereas earth-fill embankment type dams are usually built with an inclined core due to lack of necessary volume of impervious material, it was the consolidation characteristics of the embankment materials that decided the usage of this type of core. While a vertical core would be the most economical design, engineers feared that this would cause an "arching" failure of the core through horizontal cracks in its center as a result of differential settlement. One of the great benefits of inclined cores is the mitigation of this effect due to their sloped nature (US Army Corps of Engineers, 2004). This was proved to be a correct choice later on during construction, as settlement and stress measurements detected a harmless amount of arching in the core (California Deparment of Water Resources, 1974).

4.5 Stability Analyses

In order to determine the safety factor of Oroville Dam during an earthquake, a 0.1g horizontal seismic acceleration was incorporated into the conventional analyses. Depending on the initial conditions of each test, total stress or effective stress basis soil strengths were used. The calculated safety factors were well within the necessary criteria (California Deparment of Water Resources, 1974).

While Oroville Dam is not within a region of high seismic activity compared to the rest of California, extensive steps were taken to assure the structure's safety in case on an earthquake. The embankment is founded directly on bedrock, except the outer shells, which lie atop an amount of sand with greater density than that of the embankment, eliminating the possibility of liquefaction occurring at the dam's foundations. Furthermore, the zoning of the embankment provides suitable transitions between the impervious core and the outer shells. According to (California Deparment of Water Resources, 1974), the 6.7 m (22 ft.) of freeboard above normal maximum water level required for the maximum flood is more than normally required, in order to account for combinations of crest-slumping and earthquake-generated waves. (California Deparment of Water Resources, 1974)

4.6 Site Geologic Exploration

Oroville Dam is founded on an unnamed metavolcanic rock formation within the "Bedrock Series", comprised of mostly amphibolite. It is hard, dense, gray to black, fine to coarse grained, and massive in many locations, yet foliated or schistose segments are also common. The average foliation attitude strikes 12 degrees west of north and dips 77 degrees east. There are three prominent sets of joints which characterize the rock with a certain blockiness, and the depth of weathering in the rock varied greatly from location to location. In addition, fresh rock was exposed in small segments near the abutments an on the riverbanks. In the sheared zones, weathering reaches 30 m (100 ft) in depth (California Deparment of Water Resources, 1974); (US Army Corps of Engineers, 1970).

Underground geologic exploration was conducted by the US Army Corps of Engineers and the Department of Water Resources. All required tests were complete by 1959. These included exploratory adits, core borings, seismic surveys, and special tests, such as x-ray diffraction, studies of blasting, bedrock-stripping procedures, and measurement of groundwater levels and spring flows. (California Deparment of Water Resources, 1974)

4.7 Foundation Excavation and Grouting

The excavation criteria for the foundation of the main dam embankments were as follows:

Concrete Core Block: Sound hard rock, fresh to slightly weathered, unstained to slightly iron-stained fractures.

Embankment Core Trench: Sound rock meant to become impervious after grouting. Seams and shear zones were excavated to a depth equal to their width. Any irregularities in the rock were removed to allow compaction of the core.

Embankment Shells: Weathered rock with a definable strength equal to that of the materials placed on top of it.

Next, grouting for the main embankments consisted of a 61 m (200 ft.) maximum depth single cement grout curtain. This was achieved through 12.2 (40 ft.) foundation drain holes with a maximum of 24 m (80 ft.) of spacing between them, angling downstream from the grout curtain into the grout gallery. Finally, in order to improve the strength of fractured areas within the core trench foundation. slush and shallow blanket grouting were additionally provided. (California Deparment of Water Resources, 1974)

4.8 Core Block and Grout Gallery

In order to avoid attempting to compact Oroville Dam's impervious core above the irregular, eroded surface of the Feather River banks, a lean concrete core block was created. Comprised of 18 monoliths, this block has a volume of 216,000 m^3 (283,000 cubic yards) and a flat top at elevation 76.2 m (250 ft.). This structure allowed the quick placement of 1,500,000 m^3 of embankment material upstream in advance of the 1964 construction season and the building of the 122 m (400 ft.) high cofferdam, which would be later incorporated into the main dam.

Under Oroville Dam's core, starting from the core block, lies a reinforced concrete grout gallery. It extends up to the approximate elevation of 238 m (780 ft.) up the right abutment and up to 250 m (820 ft.) elevation up the left abutment. Under the core, the average depth of the gallery is 4.5 m (15 feet) and the width is around 3 m (10 ft.). (California Deparment of Water Resources, 1974)

4.9 Diversion-Tailrace Tunnels

The alignment of the two diversion tunnels was selected in order to comfortably bypass the dam work site, as well as provide the ability to be later connected to the underground powerplant for use as tailrace tunnels. The intake elevations for these tunnels are 64 m (210 ft.) and 70 m (230 ft.), whereas the outlets are at 55.47 m (182 ft.) and 63 m (207.5 ft.) respectively. Diversion Tunnel No.1 was completed by November 1963, and the second tunnel was ready by November 1964. These two tunnels together with the 122 m (400 ft.) high cofferdam were able to withstand the 1964 flood mentioned previously. After they were no longer needed for diversion, each of the upstream ends of these tunnels were plugged. Tunnel No. 2 was plugged during August 1966 and Tunnel No. 1 in November 1967, essentially marking the beginning of filling Lake Oroville. (California Deparment of Water Resources, 1974)

4.10 River Outlet

The river outlet is located next to the upstream plug in Diversion Tunnel No. 2. It is comprised of two 72-inch conduits cast into the plug, and stream releases are controlled by two 72-inch spherical shutoff valves. These valves use an electrohydraulic activation and control system, with two available operations: Standard settings for opening and closing the valve from the valve chamber at elevation 71 m (233 ft.), and an emergency remote closing of the valve from the equipment control chamber at elevation 88 m (290 ft.).

4.11 Spillway

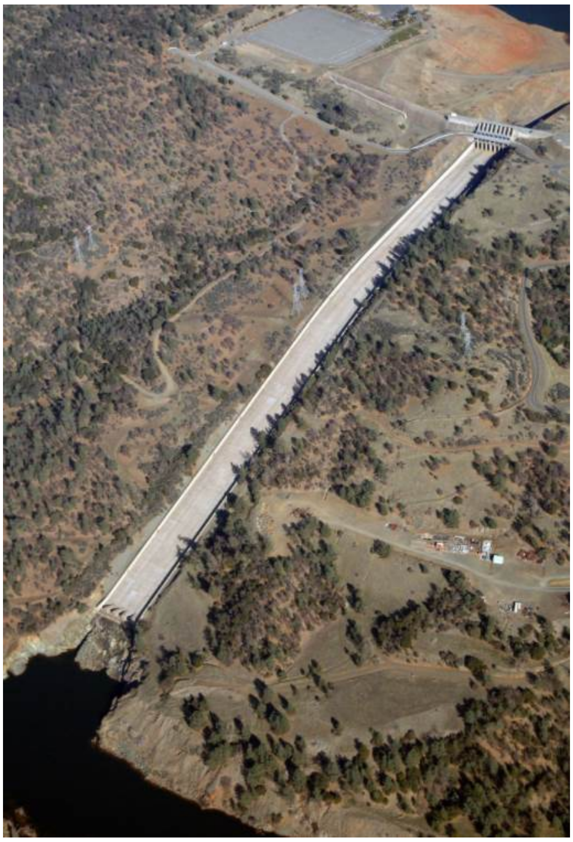


Image 3. Aerial view of the Oroville Spillway, 16th January 2014. Source: Paul Hames / California Department of Water Resources (2017)

Oroville Dam's spillway is located on a natural ridge adjacent to right abutment of the main embankments. It consists of two independent structures, a combined flood control outlet and an emergency weir. The former consists of an unlined approach channel with walls in a such a way as to make flows smoothly transit into an outlet passage, a headworks, and a concrete lined chute, approximately 929 m (3050 ft.) in length. The headworks structure is comprised of eight top-seal radial gates, 17.78 cm (7 inches) thick and 5.18 m (17 feet) wide by 10.06 m (33 feet) high. At the end of the lined chute, chute blocks help absorb some of the energy from the outgoing flow before it pours into the Feather River.

The main concept behind designing the flood control outlet was to limit Feather River flow to 5,094 m^3/s (180,000 cfs) in the occurrence of a flood event known as the Standard Project Flood (SPF). The definition of this flood is given by (American Meteorological Society, 2012) as "*the discharge expected to result from the most severe combination of meteorological and hydrological conditions that are reasonably characteristic of the geographic region involved*". For Oroville Dam, the peak inflow of the SPF was estimated at 12,700 m^3/s (450,000 cfs), and has a recurrence interval of 450 years (California Deparment of Water Resources, 1974). This flood is estimated to cause a runoff of approximately 850 m^3/s (30,000 cfs) at the confluence of the Feather and Yuba rivers located 56 km (35 miles) downstream of Oroville Dam. In order to meet this criteria, the flood control was designed for a 4,245 m^3/s (150,000 cfs) controlled release, and a flood control reservation volume of 925.11 hm^3 (750,000 acre-feet) was deemed necessary. This volume is also mentioned in the official manual for flood control operation of Oroville Dam (US Army Corps of Engineers, 1970); (California Deparment of Water Resources, 1974).

According the flood control manual (US Army Corps of Engineers, 1970), the additional following restrictions are applied to flows from the main spillway:

1) Any water stored in the designated flood control space should be released as quickly as possible, according to a given flood control diagram. Flows from Feather River should not exceed 150,000 cfs (4,245 m^3/s).

2) During extreme flood events, releases greater than 150,000 cfs may be required in order to minimize uncontrolled spillway discharges.

3) Releases from Oroville Dam are not to be increased more than 280 m^3/s (10,000 cfs) or decreased more than 140 m^3/s (5,000 cfs) in any given 2-hour period.

In the event that an upcoming flood is forecasted to be greater than the SPF, releases through the flood control outlet may be increased above the designated 4,245 m^3/s (150,000 cfs), but may not exceed 90% of the incoming inflow (California Deparment of Water Resources, 1974). The design capacity of the main spillway is 7756 m^3/s (277,000 cfs) (US Bureau of Reclamation, 1965), so if only this structure were to be used combined with the 90% inflow restriction, the theoretical maximum incoming inflow that can be accommodated is 8618 m^3/s (307,778 cfs), which is significantly lower than the peak flood inflow of the SPF.

According to (California Deparment of Water Resources, 1974); (US Bureau of Reclamation, 1965), the combined capacity of the main and emergency spillways is 17,472 m^3/s (624,000 cfs), which corresponds to a peak inflow of 20,160 m^3/s (720,000 cfs). The event that would cause this inflow is named the Probable Maximum Flood, or PMF. (American Meteorological Society, 2012) defines it as the *"flood that can be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in a region"*. The PMF for a given basin is usually derived from the Probable Maximum Precipitation, or PMP. In turn, the AMS definition for this term is *"theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of year"*. There are various methods to estimate the PMP and resulting PMF for any given region, but the concept that there exists a theoretical upper limit of precipitation depth for a given location at certain time of the year has fundamental flaws (Koutsoyiannis, 1999). This particular aspect will be analyzed further later. In any case, given the known design capacity of the main spillway, this would set the design capacity of the emergency spillway to approximately 9,900 m^3/s (350,000 cfs) in order to meet the combined outflow required by the PMF.

When the Oroville spillway was still being designed, various different types were studied, and the results of this project can be found at (US Bureau of Reclamation, 1965). The final structure consisted of the separated flood control outlet and emergency spillway seen today. During its construction, design specified that the concrete meant for the spillway chute, weir, and flood control structures was to obtain a strength of 3,000 psi in 28 days, whereas lower portions of the flood control outlet and concrete directly below the prestressed trunnion anchorages were specified 28-day strengths of 4,000 and 5,000 psi respectively. The structural steel for the radial gates is ASTM Designation A441, whereas secondary members and beams are built out of A36 (California Deparment of Water Resources, 1974).

Concerning the chute of the main spillway, with the exception of the end section containing chute blocks, all walls are cantilever type between 16 and 34 feet in height (4.8 and 10.36 m respectively). From a structural standpoint, they are independent of the slabs comprising the chute invert. The invert slabs have a minimum thickness of 380 mm (15 inches), are anchored to rock with grouted anchor bars, and are provided with a system of underdrains.

The terminal structure at the chute's end was designed with the prospect of diffusing larger flows, and is keyed into rock foundation in order to resist the massive forces the flowing water exerts onto it. At this location, large chute blocks are mounted on the invert. These are 7 m (23 ft.) high and 13.10 m (43 ft.) tall. These separate the flow, which ends in an energy dissipation plunge pool excavated at the chute's foot, linking it to the Feather River downstream of Oroville Dam (California Deparment of Water Resources, 1974); (US Bureau of Reclamation, 1965). This energy dissipation is seen in action in the following image.

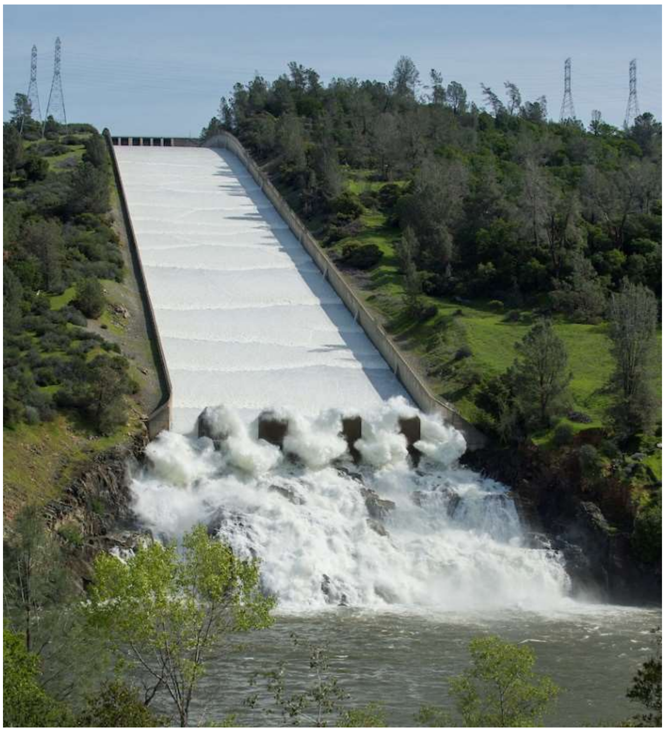


Image 4. View of the Oroville main spillway terminal structure, March 24, 2016. Flow is 140 m^3/s (5,000 cfs). Source: Kelly M. Grow/California Department of Water Resources

4.12 Emergency Spillway

On the other hand, the emergency spillway is a much simpler structure, consisting of only a concrete overpour weir with no gates. It is actually split into two sections, one 283 m (930 ft.) long on the left side up to 15.24 m (50 ft.) in height, and one 244 m (800 ft.) section to the right, which is a broad-crested weir built atop streambed excavation. According to (California Deparment of Water Resources, 1974), the area directly below the emergency spillway was not cleared of trees due to the fact that this structure would not be frequently put to use.

The grout curtain of the dam's main embankment was continued under the emergency weir's left reach, and drains are used under the downstream half. The crest of this weir on the right side rises only one foot (0.30 m) above the excavated channel, yet is keyed 2 feet (0.60 m) into the foundation. Both sections of the emergency spillway were checked for overturning and shear friction, and resulting safety factors were satisfactory (California Deparment of Water Resources, 1974). However, there seems to be no mention in official documents of the emergency spillway having ever been tested for any water flow, let alone for its rather high design capacity. The emergency spillway was also not included in final model studies conducted by the US Bureau of Reclamation (US Bureau of Reclamation, 1965).

4.13 Headworks

The headworks structure has a total length of 174m (570 ft.), rising at the same height as Oroville Dam's crest at elevation 281 m (922 ft.). The outlet passages for the main flood control outlet are built in an excavated channel, at a lower level than that of the auxiliary spillway approach. The outlet's invert is at elevation 248 m (813.6 ft.). In addition, the embankment grout curtain extends under the headworks as well, at a maximum depth of 15.24 m (50 ft.). Drain holes have been drilled beneath it into the foundation rock, and uplift pressures have been assumed to be 100 percent of reservoir head at the upstream edge, followed by a linear reduction to 33 percent and zero percent at the drains and downstream end respectively (California Deparment of Water Resources, 1974).

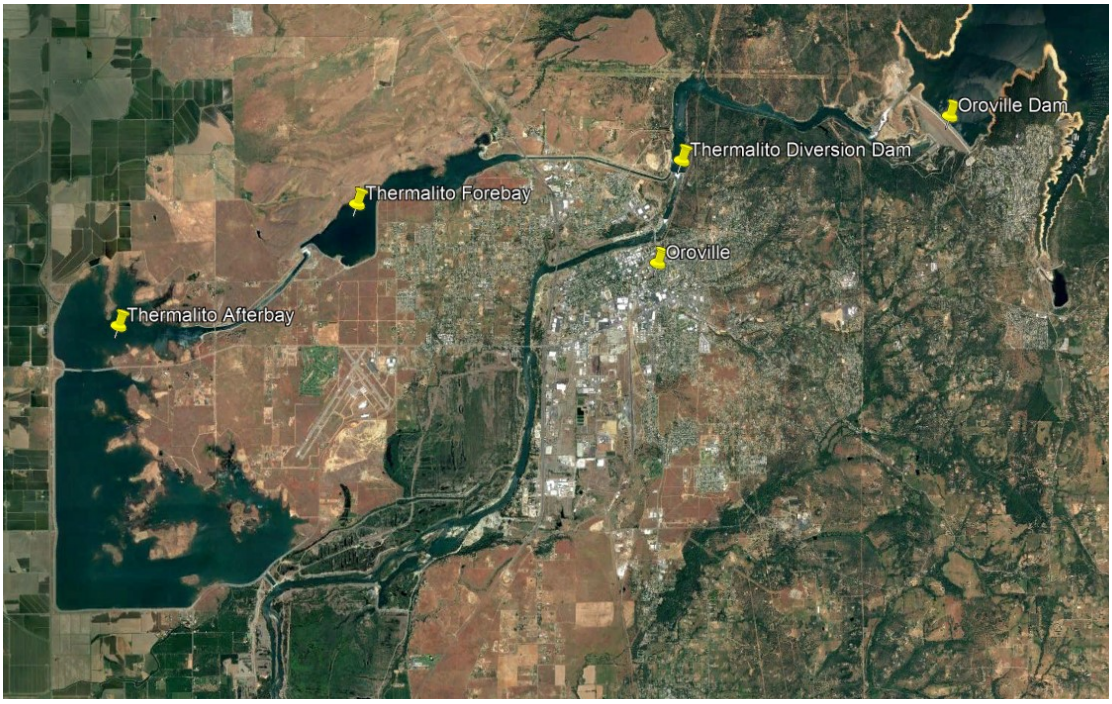
Rubber seals are attatched to all four sides of each of the eight radial gates. The bottom seal is mounted in the sill plate, closing tightly due the gate's weight. Hydrostatic pressure is applied behind the seals directly through the reservoir, with a two-way valve system built to relieve pressure when moving the gates. The side seals slide against embedded steel plates in the structure's walls. The noses of these seals are teflon-clad, with an assumed friction coefficient of 0.1.

The eight radial gates are operated by electric motor-powered cable drum hoists located on a hoist deck. This deck is comprised of 46 cm (18 inch) reinforced concrete slabs, built to support the maximum possible force caused by lifting the gates, resulting in a design load of 12 kN/m^2 (250 pounds per square foot). Each gate is operated locally or remotely from the Oroville Area Control Center. Power for the hoist operation is primarily supplied through the Edward Hyatt station power service system using a buried distribution line. If this source is not feasible, standby power can be made available through a backup 55-kW generator operated by a liquid-propane gas-fueled engine.

4.14 Thermalito Diversion Dam

Thermalito Diversion Dam is a concrete gravity structure rising 44 m (143 ft.) above streambed excavation, with a crest length of 400 m (1300 ft.). Located downstream of Oroville Dam, and just 1 mile upstream of the city of Oroville, its purpose is to divert Feather River flows to the Thermalito power generation facilities. The reservoir behind it has a total capacity of 16 hm^3 (13,300 acre-feet), rising up to an elevation of 69 m (225 ft.). It also contains an overflow section, built for a maximum capacity of 9100 m^3/s (320,000 cfs). In the event of the design probable maxium flood (PMF) the structure would be overtopped, resulting in a peak discharge of 18,300 m^3/s (646,000 cfs). The flow diversion is achieved through a special canal, which conveys up to 480

m^3/s (17,000 cfs) of water from the diversion reservoir to the forebay under normal power generation conditions, and in reverse direction under pumping operations. The forebay is comprised of an earthfill dam with a height of 22 m (71 ft.), a crest length of 4850 m (15,900 ft.), and a total storage capacity of 14.5 hm^3 (11,800 acre-feet). A powerhouse below this dam is supplied with three reversible pump turbines with a total power generation capacity of 115 MW. Below this second dam, a unlined tailrace channel conveys releases to a third reservoir, the Thermalito Afterbay. This final structure consists of an 11 m (37 ft.) high earthfill dam impounding 70 hm^3 (57,000 acre-feet) of water at elevation 42 m (136.5 ft). From here, water is released for irrigation purposes through special outlets, or is rerouted towards the Feather River.



Map 6. Map showing the location of the Thermalito facilities and Oroville Dam. Source: Google Earth (2017).

4.15 Plans

(California Deparment of Water Resources, 1974) contains detailed plans of Oroville Dam. A selection of those directly related to this thesis can be found in Appendix B.

5. The Construction of Oroville Dam

5.1 Contractors

As stated before, Oroville Dam and necessary related facilities were accomplished by several separate contracts. A detailed schedule of construction beginnings and ends for each part of the dam, as well as the winning contractor can be found here (DWR, 2005).

Table 14. Chronology of the construction of Oroville Dam and related facilities.

Activity	Start of Construction	Construction Completed	Constuctor
DAM, RESERVOIR & POWER FACILITIES			
Diversion Tunnel No. 1 (Spec. 61-05)	18-Aug-61	16-Jan-64	Frazier Davis Const. Co.
Palermo Outlet Works (Spec. 61-15)	11-Dec-61	03-Jun-63	Morrison-Knudsen Co.
Oroville Dam (Spec. 62-05)	13-Aug-62	29-Jun-68	Oro Dam
Construction of Construction Headquarter (Spec. 62-27)	16-Nov-62	12-Dec-63	A. Teichert & Son
Furnishing & Installing Turbines and Pumps (Spec. 63-05)	17-Jun-63	18-Feb-71	Allis-Chalmers Manufacturing Co.
Hyatt Pumping-Generating Plant (Spec. 63-06)	24-Jun-63	16-May-67	McNamara Corp. & G.A. Fuller Co.
Quincy Rd. Relocation Oroville-Forbestown (Spec. 63-35)	03-Jan-64	08-Sep-65	Piombo Construction Co.
Thermalito Turbines, Pump – Turbines and Governors (Spec. 63-39)	25-Feb-64	17-Mar-70	Allis-Chalmers
Furnishing 114 Inch Spherical Valves (Spec. 64-13)	30-Mar-64	16-May-69	Baldwin-Lima-Hamilton
Furnishing & Installing Generator&Motor/Generator (Spec. 64-16)	03-Jul-64	04-May-72	Westinghouse Corp.
Thermalito Power Canal Relocation (Spec. 64-31)	30-Oct-64	10-Nov-65	Osborn Construction Co.
Thermalito Pumping-Generating Plant (Spec. 64-37)	04-Dec-64	13-Feb-69	Guy F. Atkinson Co.
Furnishing Radial gates and hoists for Thermalito Diversion Dam (Spec. 64-43)	15-Dec-64	16-Nov-66	Berkeley Steel Const. Co., Inc.
Clearing Oroville Reservoir site (Spec. 65-05)	12-Apr-65	08-Jun-67	C.J. Langenfelder & Son, Inc.
Intake Trashracks and Shutters (Spec. 65-11)	30-Apr-65	22-Dec-67	Michel & Pfeffer Iron Works, Inc.
Furnishing / Installing One Generator and Three Motor – Generators Thermalito Pumping Plant (Spec. 65-02)	03-Jun-65	03-Sep-69	Allis-Chalmers
Oroville Dam Spillway (Spec. 65-09)	**25-Jun-65**	**12-Mar-68**	**Oro Pcfc Cnst & G. Farnsworth Cnst. Corp.**
Feather Falls Rd. Relocation South Fork Feather River Bridge and Roadway (Spec. 65-26)	10-Aug-65	24-Jan-68	Rthchld, Rfin & Wirck, Inc. & Piombo Const. Co.

Power Transformer-substation Transformer & Lighting (Spec. 65-31)	25-Aug-65	18-Aug-69	Moloney Electric Co.
Thermalito Power Canal (Spec. 65-37)	07-Oct-65	31-Oct-67	Morrison-Knudsen Co., Inc.
Thermalito Forebay and Afterbay (Spec. 65- 27)	25-Oct-65	19-Apr-68	Guy F. Atkinson Co.
Falls Road Relocation Feather (Spec. 65-23)	23-Dec-65	28-Sep-67	O.K. Mittry & Son
230 KV Power Circuit Breakers (Spec. 65-38)	29-Dec-65	25-Feb-69	General Electric Co.
Completion of Hyatt Pumping-Generating Plant (Spec. 66-32)	31-Aug-66	23-Jun-69	Wismer & Becker
Oroville-Thermalito Control system (Spec. 66-44)	17-Oct-66	18-May-72	Philco Corp.
Oroville Operation & Maintenance Center (Spec. 66-52)	23-Jan-67	15-Apr-68	Christensen & Foster
Oroville-Thermalito Bus Lines (Spec. 67-01)	06-Feb-67	29-Aug-68	Wismer & Becker Contracting Engineers
Completion of Penstock Intake – Left Abutment (Spec. 65-52)	25-Jan-68	14-May-68	Yuba Consolidated Industries, Inc.
Thermalito Fish Rearing Raceways (83-06)	25-Apr-83	20-Mar-84	Kaweah Construction Co
Powerplant-furnishing Turbine-Generator Governor (Spec. 84-19)	01-Aug-84	03-May-88	Hitachi America, Ltd.
FISH FACILITIES			
Interim Facilities Feather River Hatchery (Spec. 62-01)	16-Mar-62	19-May-64	Frazier-Davis Construction Co.
Feather River Fish Hatchery (Spec. 66-18)	16-May-66	18-Dec-67	Peterson & BrownEly

Bid amounts and final costs for each facility can be found at (California Deparment of Water Resources, 1974). For reference, the total contruction costs for the main Oroville Dam and the spillway respectively were $135,336,156 cost $13,702,871. Adjusting for inflation starting from the year 1962, these prices today would be $1,106,077,307 for the dam and $111,991,023 for the spillway. Total cost for the project is an estimated $438 million, or $3 billion in today's currency. (California Deparment of Water Resources, 1974); (Coinnews Media Group LLC., 2017). At the time, the main work was the largest civil engineering contract (in dollars) in the United States.

5.2 Diversion Tunnel No. 1

Image 5. October 12, 1961. The groundbreaking ceremony for Oroville Dam, including the first blast for the construction of Diversion Tunnel No. 1. Source: DWR.

Open-cut excavation at the intake portal began on October 2, 1961. On November 14 of the same year, a 30.5 cm (12 inch) wide crack developed near the centerline of the tunnel, and the partially completed portal soon collapsed. Measures to increase slope stability and reinforce the local rock on were put into place. An umbrella built out of arch ribs was installed in wall plates, reinforced with timber cribbing, and then covered with dredger tailings. These actions allowed tunnel excavation to resume on January 9, 1962. Open-cut excavation of the outlet portal started in July 10, 1962, and was completed with no issues.

In order to prevent flooding of the tunnel from the downstream entrance during the precipitation-heavy 1962-63 winter, a 11.6 m (38 ft.) long section was left unexcavated. However, two floods occurring during that water year, caused damages at the work site. The first, on October 1962, had a peak of 3,850 m^3/s (136,000 cfs), overtopped the upstream levee and flooded the tunnel from that end and destroyed the downstream Bailey Bridge. The second, on January 1963, with a peak of 5,400 m^3/s (191,000) cfs, indundated the area and damaged the replacement bridge.

For the excavation of the diversion tunnel, the top heading method was used, and the excavation of the tunnel was made possible through two three-deck jumbos on truck chassis. The average rate of advance was 6.25 m (20.5 ft.) per 24-hour 3-shift day. By November 1963, construction was completed and the tunnel was put into use. (California Deparment of Water Resources, 1974)

5.3 Palermo Outlet Works

Work at the Palermo Outlet works began by diverting drainage from the intake and outlet portals. The rock encountered at the face of the upstream portal was moderately to strongly weathered, moderately hard, blocky amphibolite, additionally covered in soil at slopes of 1.5:1 and 2:1. Due to the blocky nature of the rock at this point, additional reinforcement was required. On the other hand, the downstream portal was excavated with a 1.5:1 slope in fresh to slightly weathered, hard, strongly jointed amphibolite with a much thinner cover of soil. Both portal cuts were achieved with a ripper-equipped tractor and a minimal amount of blasting, to loosen large blocks. Inside the tunnel, the amphibolite encountered was generally fresh, moderately blocky or jointed. In three zones of sheared weathered rock within the tunnel, additional support was required, and accomplished using W4X13 steel beams. The tunnel was driven using rail-mounted equipment, using three hydraulic drifters mounted on a jumbo for face drilling. Finally, additional grouting was done from within the tunnel to avoid additional grouting adjacent to the outlet works during curtain grouting of the main dam.

5.4 Feather River Diversion and Foundation Dewatering

Diversion of the Feather River and foundation dewatering were carried out in four stages.

During Stage 1, the river remained in its natural channel while the lower lifts of core blocks 1 through 7 and 9 were being placed.

Stage 2 began in July 18, 1963. The river was diverted through block 8 with the use of an earth dike across the channel upstream of the core block, and a second dike downstream for foundation dewatering.

In Stage 3, the river was diverted through a sluiceway built along block 12, permitting the dewatering of block 8 and resuming construction by September 4, 1963.

Finally, the last diversion was through Tunnel No.1 on November 15, 1963.

5.5 Core Trench and Grout Gallery Excavation

Grout gallery excavation was generally within sound rock, so drilling and blasting was required to proceed. For the core block, excavations continued up to hard, fresh rock which would be as impervious as required for the final foundation after grouting. Water jets helped provide cleanup after the necessary procedures. After the stripping concluded, a very large amount of material had to be removed, approximately 182,000 m^3 (238,000 cubic yards) in total. This amount was significantly higher than expected, and the overall final excavation costs had to be adjusted, according to the following table.

Table 15. Comparison between excavation bid estimates and the final paid estimates. Source: DWR (1974).

	Bid Estimate		**Actual Pay Estimate**	
Class of Excavation	*cu. yards*	*m^3*	*cu. yards*	*m^3*
Stripping	2,860,000	2,186,627	4,844,300	3,703,733
Core Trench	690,000	527,543	1,112,200	850,338
Grout Gallery	30,300	23,166	55,500	42,433

The fact that the final amounts of material to be removed ended up much higher than the original estimates was due to design specifications. All excavations were required to reach solid rock which would be of an appropriate quality to build foundation upon. However, the rock excavated at the Oroville Dam site proved to be weathered more extensively than originally expected, and thus it was necessary to remove a much larger quantity.

Furthermore, several slides occurred in locations where excavations was deemed complete. The largest occurred on the left abutment of the dam, just upstream of the core trench. A large section of rock approximately 46 m (150 ft.) in length and 12 m (40 ft.) in height detatched from the hillside in a broken mass. Heavy rains during the December 1964 flood saturated the material, and possibly filled cracks within the weathered rock formation with water. This resulted in a much more extensive slide, removing nearly 75,000 m^3 (100,000 cubic yards) of material. The top of the slide was at the approximate elevation of 213.36 m (700 ft.). (California Deparment of Water Resources, 1974)

5.6 Core Block Construction

The core block was placed parallel to the dam axis. Wood forms were used for concreting on foundation rock, until a suitable height was reached for the use of steel cantilever panel forms. These forms were set with the use of a hydraulic crane, operating on top the blocks and moving between them with the use of a 25-ton capacity high line. On the other hand, the concrete was placed by way of a 6 m^3 (8 cubic yard) placing bucket carried by a tramway, resting on a 426 m (1,400 ft.) long cable, supported by two rail-mounted steel towers operating along a travel of 122 m (400 ft.). This system allowed all points of the core block construction area to be reached.

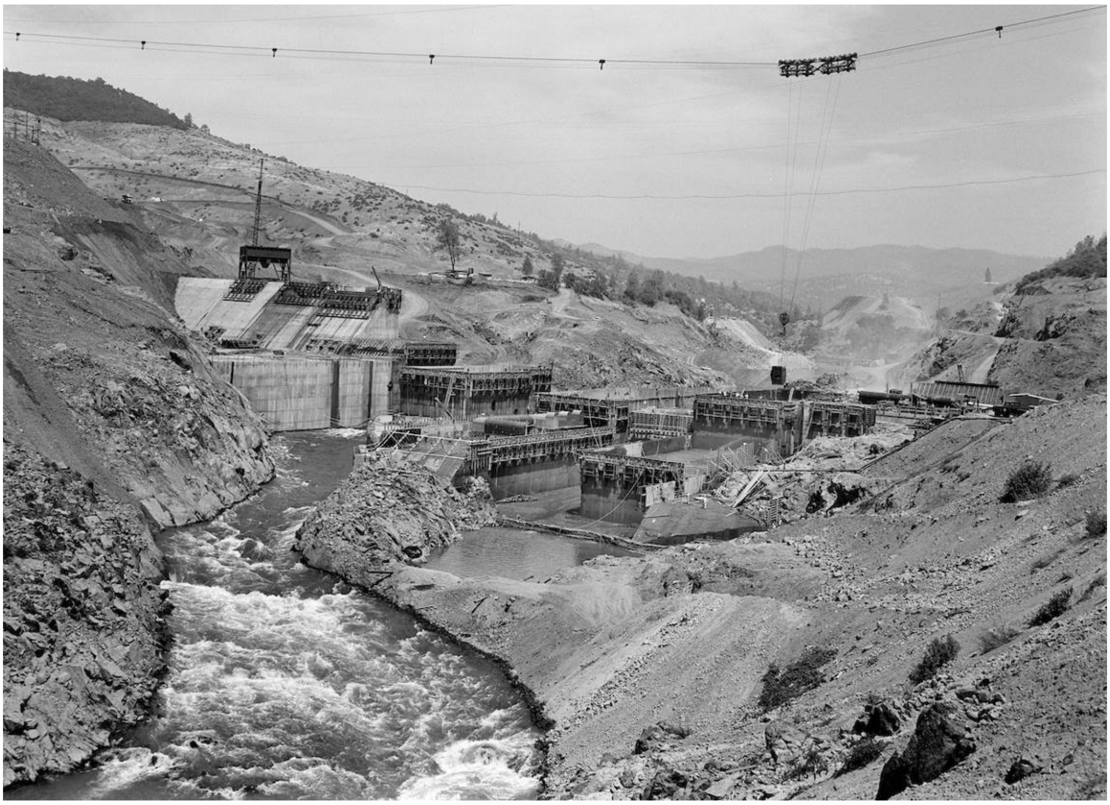


Image 6. View of the Oroville Dam core block construction work site. Photo taken August 30, 1963. Sources: Gene Russell, DWR

In addition, the concrete used for the core block was 150 mm (6-inch) maximum aggregate mixed with pozzolan, with a required maximum temperature of 10 ºC (50 ºF). Ice was primarily used to achieve the necessary cooling levels for the aggregate, following a complicated, yet ineffective cooling plant project. The concrete was mixed in a separate batch plant, containing three 3 m^3 (4 cubic yard) mixers, then transferred to the work site with the use of rail-mounted cars, and placed with the aforementioned placing bucket system. (California Deparment of Water Resources, 1974)

5.7 Grout Gallery Construction

Construction of the grout gallery was scheduled to begin in December 1964, but heavy rains slowed down the process significantly. However, work continued regardless, and the gallery was completed within schedule. Concrete for the gallery was initially transferred from the core block plant and from the work site of the downstream Thermalito Diversion Dam, until the completion of the grout gallery plant in October 1964, which contained one 2.3 m^3 (3 cu. yard) mixer. A hopper with the same volume was located directly under it, allowing the temporary storage of concrete until it was transferred to the construction area with the use of trucks. Placement was achieved with two 0.76 m^3 (1 cu. yard) buckets operated with the use of a truck crane. (California Deparment of Water Resources, 1974)

5.8 Main Dam Embankment Construction

Placement of Oroville Dam's main embankment began as early as September 1963, in sections upstream of the core block. Initially, it was not planned to exceed elevation 88 m (290 ft.) prior to April 1, 1964. However, the exceptionally dry winter of 1963-64 allowed work to continue at a faster pace than expected, resulting in the placement of additional fill on the right abutment that exceeded the original scheduled elevation. It was this action that allowed the quick placement of the cofferdam at elevation 184 m (605 ft.) in time for the winter of 1964-65. This elevation was reached less than a month before the flood of December 1964, which was the most severe flood ever recorded at that time. With a peak of 7,080 m^3/s (250,000 cfs), it would have caused much more severe damages than the previous flood of 1955, which cost 38 lives and $100,000,000 in total. However, the expedient construction of the cofferdam allowed routing of the 1964-65 flood through the diversion tunnels, resulting in only minimal property damages downstream. (US Army Corps of Engineers, 1970) (California Deparment of Water Resources, 1974)

Zone 4, the impervious upstream core, (see Figure 10), was constructed using fine material taken from the stipping operation on the main abutments. Compaction was achieved with a pneumatic roller.

Zone 3 is comprised of coarse dredger tailings including sound rock, with a specific gravity approaching 2.9. Bottom-dump trucks deployed the material at the work site in 18 m (60 ft.) long rows, then spread by rubber-tired bulldozers. Compaction was achieved using a towed triple vibratory roller, and the design specifications required a maximum of 61 cm (24 inches) of lift thickness.

Zone 4A is a special compressible zone located just upstream of the core block. It was built to compess horizontally due to high lateral soil pressure as a result of base spreading during the 1964 construction period. Only equipment travel was used for compaction, advanced methods would produce higher compaction levels than required.

Construction material for Zones 1 and 1B, the impervious cores of the main Oroville Dam and the cofferdam respectively, were deposited on a moistened surface in 30 m (100 ft) long rows. Additional water was added using sprays to achieve the required moisture levels. The material was then compacted four times using 100-ton pneumatic rollers or an appropriately loaded ballasted truck for areas inaccessible with the first method, and hand operated compactors were also used for any remaining areas that couldn’t be covered otherwise.

Zone 2, a transitory zone between the impervious Zone 1 and the coarse Zone 3, is mostly comprised of gravel and sand, placed using a similar method as that of Zone 3, with the exception of the required lift thickness, which was 38 cm (15 inches) after compaction.

Zone 5A, a horizontal drain, was constructed at elevation 72 to 75 m (235 to 245 ft.) from downstream Zone 2 to the downstream face. In addition, Zone 5B is a vertical drainage zone 6 m (20 ft.) wide, built directly downstream from Zone 2. These zones were added after embankment construction had already begun, in order to ensure that the downstream portion of Oroville Dam would remain dry, after concerns were raised regarding fines in the pervious material which was being delivered to the work site. Compaction was achieved with the same methods and specifications as Zone 3.

Finally, zones of riprap were placed on Oroville Dam’s faces. On the upstream face, material was added between elevations 184 to 281 m (605 to 922 ft.), comprised of rock graded up to 0.7 m^3 (1 cu. yard) in size. On the other hand, riprap for the downstream portion was placed at the toe, at the face of the mandatory waste area, and contained rock fragments ranging from 0.35 to 1.5 m^3 (0.5 to 2 cu. yards) in volume. Most of this riprap rock originated from excavations in the spillway area, and was hauled directly from there. (California Deparment of Water Resources, 1974)

5.9 Diversion Tunnel No.2

The second diversion tunnel was initially excavated from the outlet portal up to within 16 m (54 ft.) of the inlet, in order to avoid damage from possible floods. For the outlet structure, open cut excavation began on January 1963. The rock on the left channel wall was deemed unsuitable, and thus several reinforcemenent countermeasures were installed. The slope of the left channel was adjusted to 1.5:1, and the left wall was bolted with expansion-shell groutable rock bolts. Excavation for the inlet portal was conducted in a similar manner to that of the outlet, but the rock at this location was extremely weathered. Therefore, an additional exploratory 3.3 by 3.3 m (11 by 11 ft.) crown drift was driven through the 16 m (54 ft.) wide plug that had been left in for flood protection. A small crack was observed over the portal, thus resulting in a relocation of the inlet structure's face, as well as the installation of several 5 m (15 ft.) rock bolts.

Tunnel excavation procedures were similar to those for Diversion Tunnel No.1, except that the invert was concreted using a slip form, and was placed downstream of the outlet structure. Additional measures were implemented in order to accommodate the tunnel's use as a tailrace tunnel later on as well. Construction was complete by November 1964, and this tunnel remained in service for the winters of 1964-65 and 1965-66. During the summer of 1966, Diversion Tunnel No. 2 was closed permanently. (California Deparment of Water Resources, 1974)

5.10 River Outlet Works

The river outlet works was installed by November 1967 and put into service just after the closure of Diversion Tunnel No. 1. The operating center for the outlet is a control cabinet supplied with 480-volt 3-phase power by the Hyatt Powerplant contractor. Controllable through this system are lighting fixtures for the grout gallery, river outlet access tunnel and control chamber, and river outlet valve chamber. (California Deparment of Water Resources, 1974)

5.11 Spillway

Image 7. Construction of the Oroville Dam main spillway flood control outlet. Photo taken January 1967. Source: DWR

5.11.1 Clearing

Approximately 115 acres (47 hectares) of land were cleared of brush and trees to accommodate the construction of the spillway. Of those 115 acres, 40 were in the main spillway and chute area, and 75 in the vicinity of the emergency spillway. The area below the emergency spillway was not cleared (California Deparment of Water Resources, 1974).

5.11.2 Excavation

The methods mainly used to excavate the main spillway were the following: bottom loading scrapers and pushcats, a loader with cats feeding the belt and bottom-dump wagons which were used to haul the material, and two large shovels. The standard procedure included using the scrapers to excavate up to solid rock, then use the shovels to excavate the rock after it was drilled. The loader was operated in rougher terrain, as it was possible to push material into the hopper using up to eight bulldozers, then transfer that material away from the work site with the use of the bottom-dump wagons. All drilling was done by air-powered percussion drills mounted on tracks, with varying patterns according to the drilling area. The most generally used pattern was 2.4 m by 2.4 m (8 by 8 feet). Excavation was limited whenever it reached too close to structure lines, in order to avoid damaging the rock that would serve as the foundation.

Especially for the emergency spillway, excavation continued 3 m (10 feet) deeper than expected, in order to reach foundation rock that met the design criteria. This significantly delayed the excavation process. Furthermore, blasting was used for almost 90% of the chute foundation, in order to reach grade. The remaining amount consisted of the removal of several seams of clay located in the foundation, and a few areas where the slope failed. In the approach channel, overburden depth was deeper than planned, thus requiring the adjustment of its slopes from 0.5:1 to 1.5:1 to prevent the occurrence of sloughing as excavation reached the final grade. Finally, the slopes in the flood control outlet section were of a lower quality rock than initially presumed, and several large seams ran parallel with the main spillway chute. The countermeasure that was applied was the replacement of planned anchor bars with grouted rock bolts, pigtail anchors, and chain-link covering of the area's surface. (California Deparment of Water Resources, 1974).

5.11.3 Drain System

The initial spillway design included nearly vertical NX holes drilled 20 m (65 feet) into the foundation rock of headworks monoliths, and extensive perforated pipe systems on the foundation surface under the headworks, chute and higher sections of the emergency spillway weir. However, this drain system plan ended up being significantly altered during construction.

After a recommendation by the Oroville Dam Consulting Board, the original 100 mm (4-inch) diameter horizontal pipe drains under the chute were enlarged to a 150 mm (6-inch) diameter, placed in a herring-bone pattern. The collector system operating in line with the chute was also enlarged and modified so as to enhance its capacity and self-cleaning ability. These pipes remained on the foundation enveloped in gravel which is a part of the chute's reinforced concrete floor. However, it was not possible to place this type of drain pipe on the irregular rock surfaces under the headworks and emergency spillway, thus they were substituted by wood-formed square drains of an equal cross-section area. These forms were cut to fit the irregularities of the underlying rock, then left in place as concrete was poured over them (California Deparment of Water Resources, 1974).

5.11.4 Chute and Emergency Spillway Construction

Concrete placement for the Oroville Dam spillway began on January 26, 1966. Mass concrete was placed monolithically in all monoliths (except Monoliths 25 and 26 which contain the flood control gates) between the transitory section at the east end of the flood control outlet and the emergency spillway (Station 18 +30 to 20 +61.66). West of this section, mass concrete placement at the auxiliary spillway continued normally. Concrete was mixed in an on-site plant, which discharged concrete into 3 m^3 (4 cu. yard) buckets positioned on low-bot trucks and hauled to the placing area, where concrete was placed using a track-mounted crane. Forms included wooden starters, which were later supplanted by 2 m (7 ft.) high and 15 cm (6-inch) thick steel cantilever versions. An adjustable steel form was used to form the curved sections of emergency spillway Monoliths 1 through 20. The uppermost section of the ogee weir was formed with wooden forms. Furthermore, structural concrete was put in place at Monoliths 25 and 26, the approach walls of the flood control outlet, the chute walls and invert, and the terminal structure at the chute's end. Concrete at the gates of the flood control outlet was placed via track-mounted crane and conveyor belts in harder-to-reach areas.

Concrete placing for the main spillway's chute invert began on September 8, 1966. Transportation of concrete from the chute banks to the actual placement point was achieved through a system of conveyor belts, whereas it reached the work site via "bathtub" trucks, transferring concrete from the batching plant with the use of a conveyor belt hopper. Wood forms were used for the chute walls, with holes cut into them to allow for concrete vibration, whearas a slip form made out of steel beams was used for the invert. In total, 122,000 m^3 (160,000 cu. yards) of concrete were placed (California Deparment of Water Resources, 1974).

5.12 Completion of Oroville Dam and Dedication

On October 7, 1967, the main embankment fill was topped out, and on November, Diversion Tunnel No.1 was plugged, essentially marking the beginning or Lake Oroville's inundation. By 1968, most of the Oroville Dam project had been completed and was ready for use, and a special celebration ceremony was held to commemorate the event. Notable figures present in this event included Chief Justice Earl Warren, Senator Thomas Kuchel, and then Governor of California Ronald Reagan, who formally dedicated Oroville Dam to the people of California (Associated Press, 1968); (Ronald Reagan Presidential Library, 1968); (California DWR Public Affairs Office, 1990). The following quote is the final segment of his dedication.

"Here before you, is Lake Oroville. Filling to its destiny for the use of flood control, hydroelectric power, irrigation, municipal, and domestic purposes, and as one of the greatest recreational and fishery lakes in California. And off there, is the highest dam in the United States. This is a major achievement of our time, and it's with great pride, therefore, that I simply dedicate Oroville Dam and Lake Oroville to the people of California's future, who will benefit from this giant structure, and the water that it impounds. Thank you very much."

-Governor Ronald Reagan, Oroville Dam, California, May 4, 1968.

6. Significant Events Following the Completion of Oroville Dam

6.1 The 1975 Earthquake

On August 1, 1975, an earthquake with a Richter Scale magnitude of 5.7 occurred approximately 12 kilometers (7.5 miles) southwest of Oroville Dam. Operation at the Oroville Facilities continued almost without interruption through the event, with the exception of the Hyatt Powerplant, which stopped its operation for around 45 minutes. Furthermore, minor damages were detected at some of the facilities, but were repaired using standard maintenance procedures (California Department of Water Resources, 1977). A seismic event of this strength is classified as a moderate earthquake (Richter, 1935), which can cause small damages to buildings, and is felt by everyone in its area of effect.

Within 5 hours of the initial event, another twenty-nine aftershocks occurred, the largest of which had a magnitude of 4.8. More seismic events followed throughout the entirety of August, but on a much more infrequent basis, with the most severe event being assigned a scale of 5.1 (California Department of Water Resources, 1977).

Estimates for repair costs of the slightly damaged Oroville Facilities barely exceeded \$8,000. The most expensive repair was that of the Thermalito Afterbay outlet embankments, which had suffered cracking and settlements, expected to cost around \$4,500. Additionally, the chute walls at the Oroville Dam main spillway terminal structure were reported to have suffered from joint spalling, but no repairs were deemed necessary at the time (California Department of Water Resources, 1977).

Seismic acceleration data were available directly through four accelerometers placed within or adjacent to Oroville Dam, and strong-motion accelerographs located at a separate seismograph station, ORV, located northwest of the dam. For the main shock, the maximum measured accelerations were 0.09g in the vertical direction and 0.13g transverse to the river.

Following these events, the California DWR conducted an investigation of the area, and concluded that the cause of the main earthquake was an until that point unmapped fault zone lying in the Swain Ravine lineament, striking in a generally northern direction, passing near the Bidwell arm of Lake Oroville, dipping 60 degrees to the west. It is unclear whether the Oroville Dam reservoir had any impact in causing this earthquake, although reservoir-induced seismic events are not uncommon. Arguments against this cite the eight years of time delay between the initial inundation of Lake Oroville and the earthquake. However, other scientists claim that since the Feather Basin was an area of reduced seismic acitivity until that event, there is a significant link between the Oroville reservoir and the activation of older local faults (Allen, 1982); (Bufe, et al., 1976). Regardless, extensive scrutiny of the Oroville Facilities yielded no further evidence of earthquake-induced damage (California Department of Water Resources, 1977).

6.2 The 1986 Flood

Record-breaking rainstorms like the one that caused massive floods in California during the middle of February 1986 are usually caused by a phenomenon called the "Pineapple Express". This is a type of atmospheric river, meaning a relatively narrow stream of enhanced water vapor. The main characteristics of this weather pattern are an intense, persistent flow of atmospheric moisture (US Geological Survey, 2010).

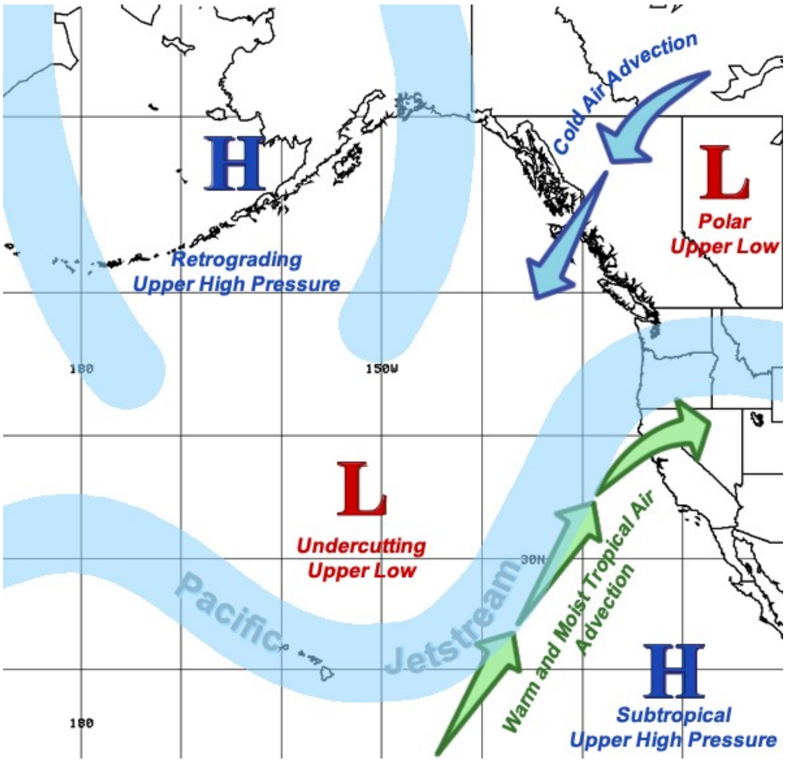


Figure 11. The general weather pattern of the 1986 "Pineapple Express". Sources: CNRFC, NOAA, 2012.

The conditions for the 1986 storm scenario were as follows: Initially, the eastern Pacific ridge retrograded towards the Aleutian Islands. This allowed a flow of cooler air from Canada to enter the atmosphere above the Pacific Ocean with a southwestern direction. As the ridge moved to the northwest, a stream of powerful low pressure systems undercut the cool flow and rushed toward the west coast of the United States. Finally, an area of subtropical high pressure to the west of Mexico helped propel this stream slightly upwards, while also contributing to it by pushing warm, moist air into the jet now directly headed towards California.

Precipitation in northern California started on February 11th, but the advection of warm, moist air from the "Pineapple Express" entered California on th 12th. This condensed, subtropical moisture hung over the Central Valley for a prolonged period of time, initiating record-high precipitation events (Meier, Ekern, Lerman, & Kozlowski, 2012). According to (California Department of Water Resources, 1997), 187 precipitation measurement stations overall reported the heaviest ever rainfall totals for any 10-day period. The precipitation records for the stations analyzed in this thesis also peak during these days, whenever measurements are available for that time period. An example of these records can be found in the following figures, for the Brush Creek (BRS) and USC00041159 stations.

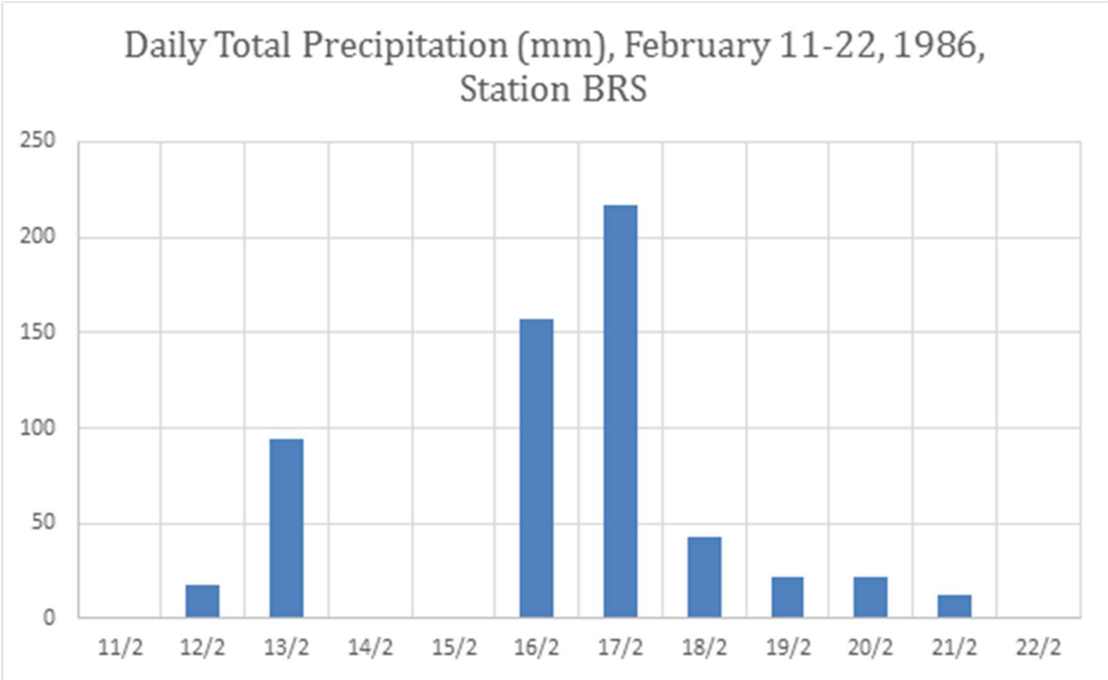


Figure 12. Daily total precipitation in mm, Brush Creek (BRS) station from the 11th to the 22nd of February 1986.

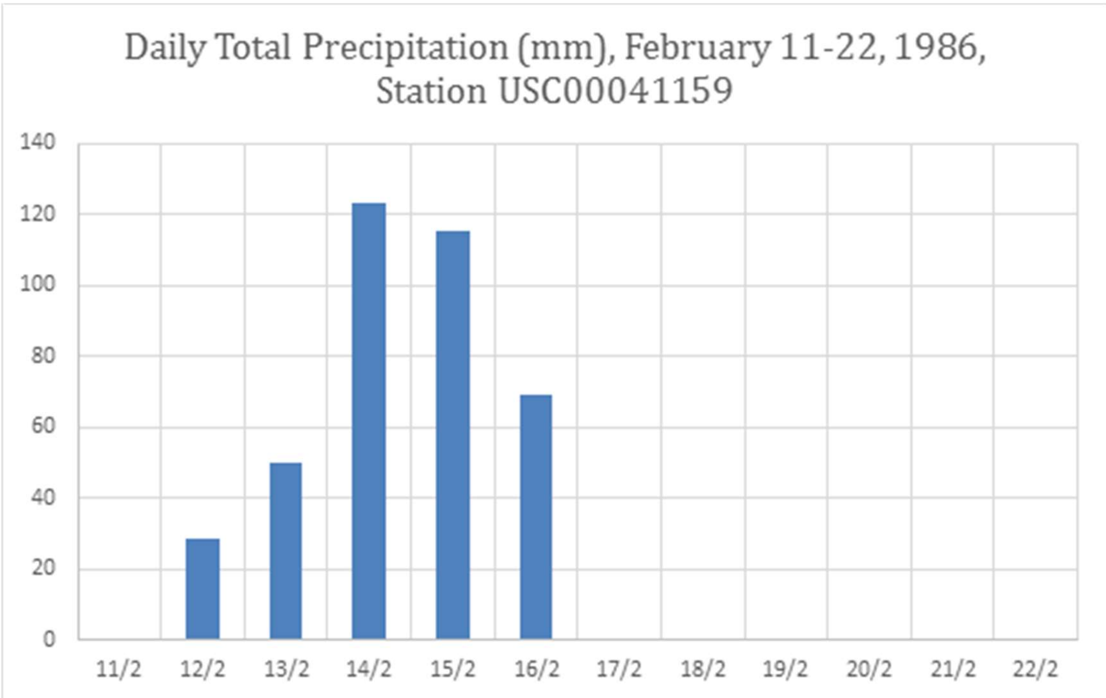


Figure 13. Daily total precipitation in mm, station USC00041159, from the 11th to the 22nd of February 1986.

Some important notes regarding these figures: For Brush Creek station, there was a gap in official measurements from the 14th to 17th of February, followed by an extremely high peak of 458.47 mm (18.05 inches) on the 18th. This is most likely caused by the measurements of the previous days being falsely added into this one, and this is confirmed later by (California Department of Water Resources, 1986); (Meier, Ekern, Lerman, & Kozlowski, 2012). However, the CNFRC reports that 10 stations throughout northern California experienced precipitations higher than 254 mm (10 inches) for the 24 hour period of February 17th.

For station USC00041159, the rain stopped on February 17th, any measurements in the following days were 0 mm (0 inches). The overall peak was on the 14th, with 123.2 mm (4.85 inches) of precipitation.

This flood was one of great significance for Oroville Dam, as it was the most intense flood to ever indundate the Feather Basin at the time. It would be the first time the spillway would be operated at the maximum allowed discharge of 4,250 m^3/s (150,000 cfs), as stated in the Flood Control Manual (US Army Corps of Engineers, 1970). This historical moment was captured in the following image.

Image 8. February 21, 1986. The Oroville Dam main spillway operating at the maximum scheduled discharge of 4,250 m^3/s (150,000 cfs). Source: Norm Hughes, DWR.

Unfortunately there are no official outflow data during that time period, but a DWR report (California Department of Water Resources, 1986) contains a detailed flow hydrograph of the 1986 flood, which can be found below.

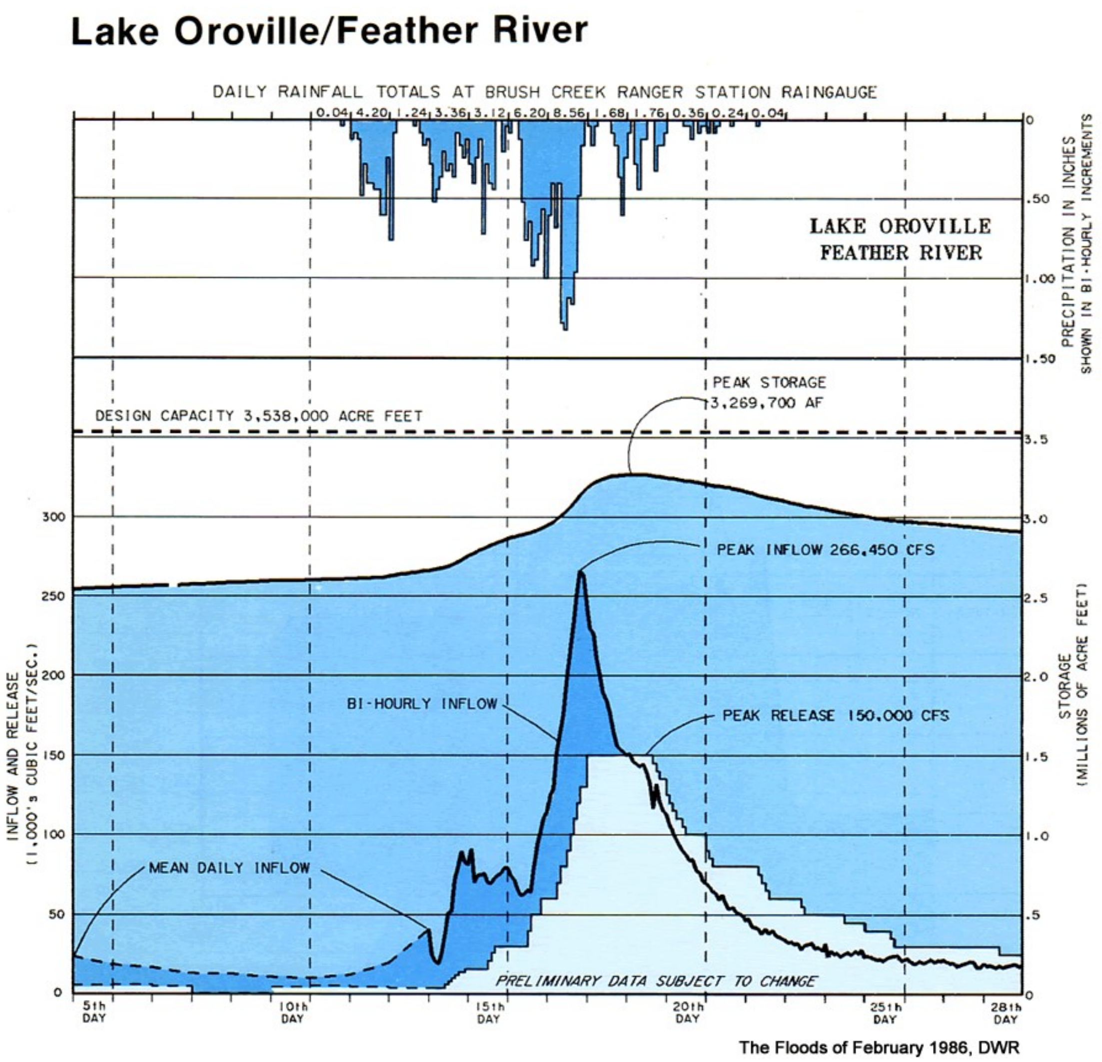


Figure 14. Flow hydrograph of the February 1986 floods. Source: DWR.

The above contains a multitude of important data, the most significant of which are as follows:

The daily total precipitation measurements at Brush Creek shown in this hydrograph reveal that the daily peak measurement of 458.47 mm (18.05 inches) given by CDEC was actually a sum of three days of precipitation, the 16th, 17th, and 18th. Furthermore, this figure contains measurements for the 14th and the 15th, which are not available in the public dataset.

The measured peak inflow into Lake Oroville was 7,545 m^3/s (264,500 cfs). This is slightly more than half of the 1967 Standard Project Flood estimate of 12,700 m^3/s (450,000 cfs), already occuring only 18 years after Oroville Dam's completion.

In addition, it is possible to compare these results to the unregulated, annual maximum flow 1-day flood data (Table 12) given by (Lamontagne, et al., 2012). Lamontagne and others calculated a 1-day maximum of 6,145 m^3/s (217,020 cfs) occurring on February 17th. By estimating the daily average discharge for the same day from the hydrograph, that discharge is approximately 6,200 m^3/s (220,000 cfs), which is near identical. Similar conclusions are drawn when comparing 3-day, 7-day, and other durations.

Finally, the lower portion of the hydrograph, shaded white, displays outflow from the main spillway. The incremental increases and decreases in discharge as requested by the Flood Control Manual (US Army Corps of Engineers, 1970) can also be seen here.

The California Data Exchange Center contains reservoir elevation and storage data for Lake Oroville during the February 1986 floods. (California Department of Water Resources, 2017). From this data, it is possible to compare the flow hydrograph to the CDEC data set.

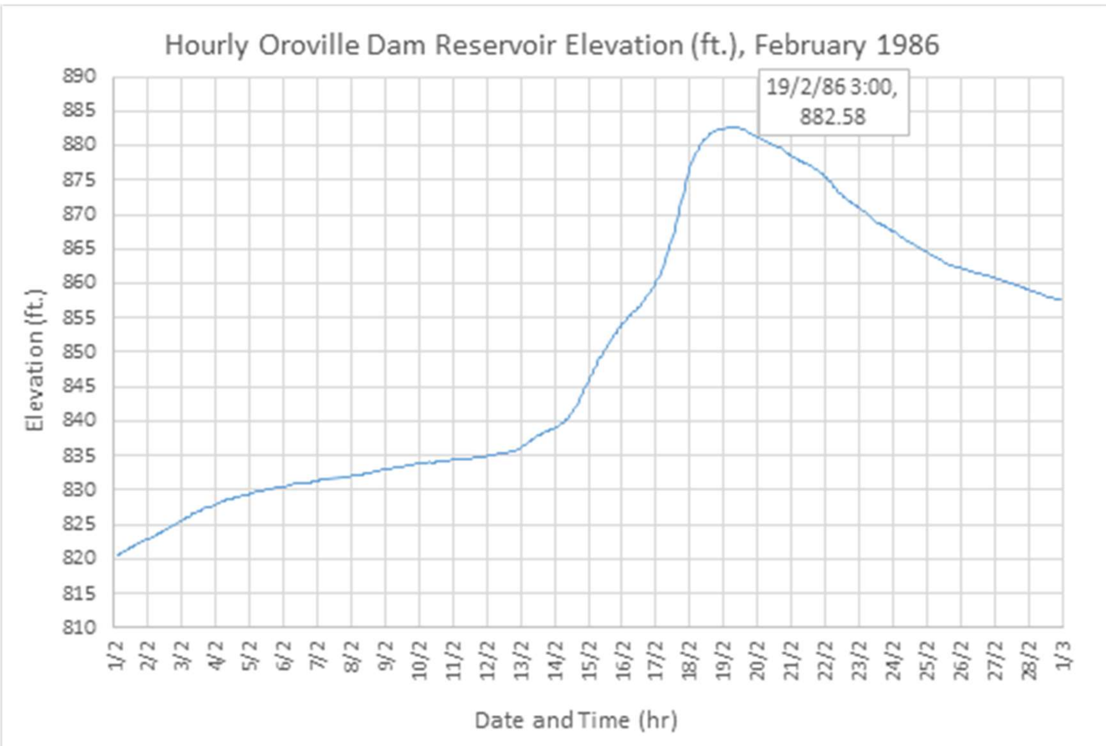


Figure 15. Hourly Oroville Dam Reservoir Elevation (ft.), February 1986.

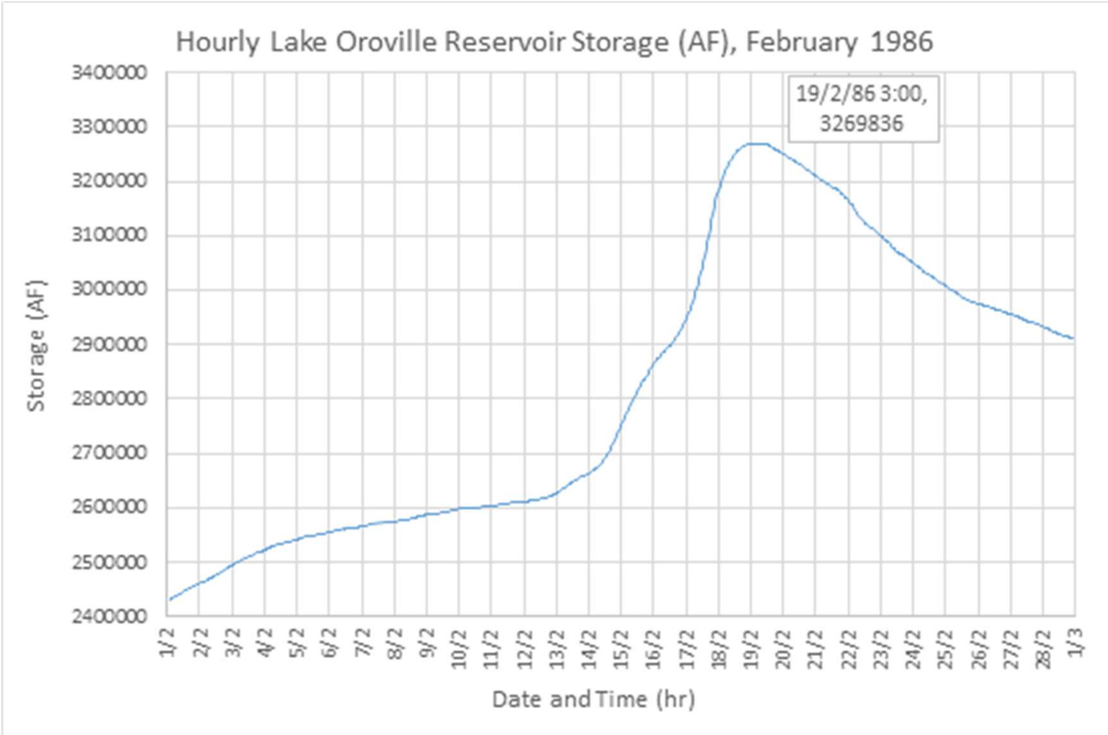


Figure 16. Hourly Oroville Dam Reservoir Storage (acre-feet), February 1986.

As is clear from the figures above, the CDEC data set provides similar results to the flow hydrograph from (California Department of Water Resources, 1986). Below are the same figures in SI units.

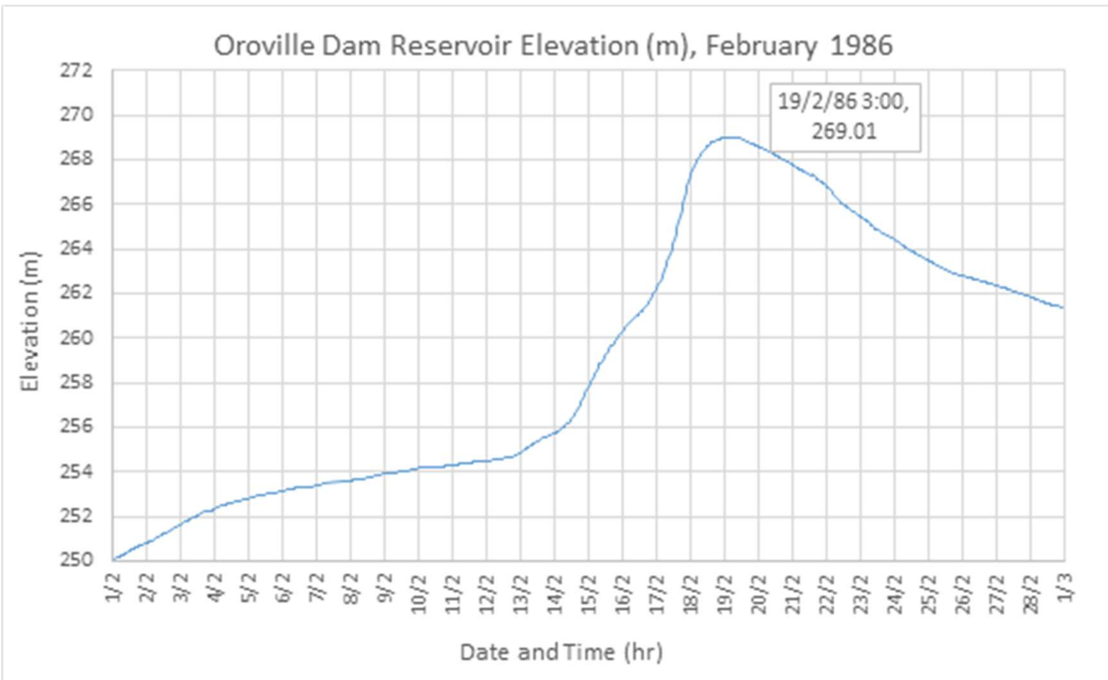


Figure 17. Hourly Oroville Dam Reservoir Elevation (m), February 1986.

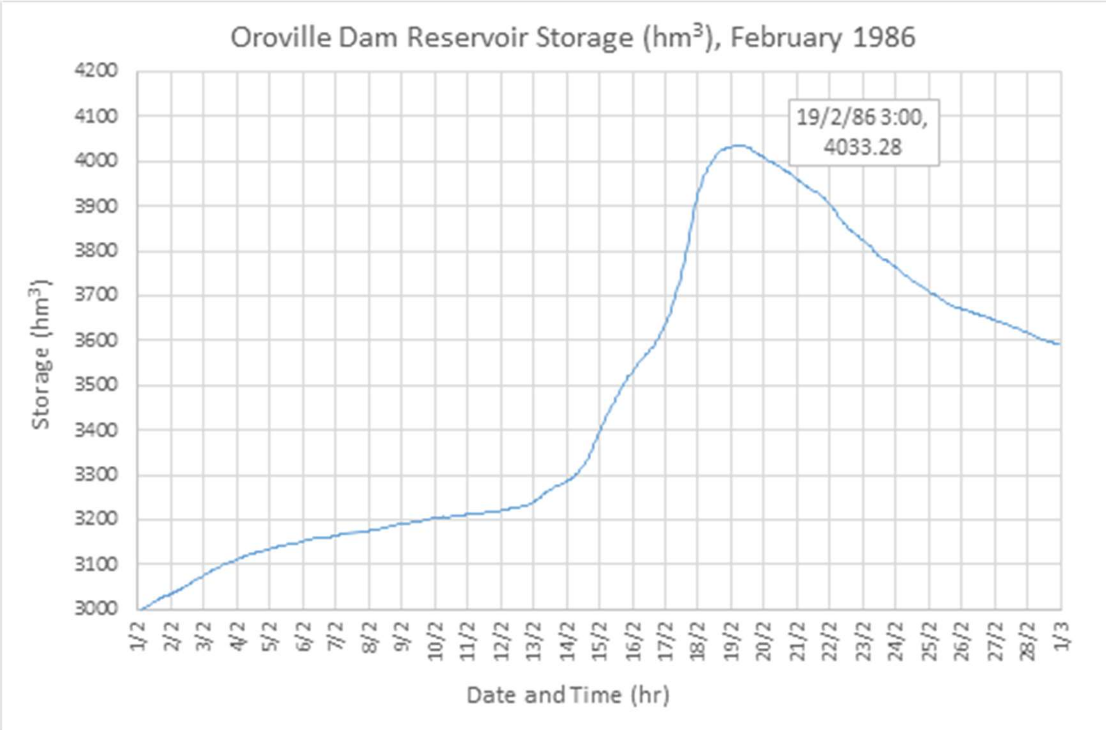


Figure 18. Hourly Oroville Dam Reservoir Storage (hm^3), February 1986.

According to the CDEC data set, the peak storage occurred at 3:00 AM on February 19, 1986 and was 4,033.28 hm^3 (3,269,836 AF), rising Lake Oroville's surface elevation to 269.01 m (882.58 ft.).

The flood control manual for Oroville Dam (US Army Corps of Engineers, 1970), specifies that 925 hm^3 (750,000 acre-feet) of storage are to be set aside for flood control before the start of every wet season. Since the maximum operating surface elevation for Oroville Dam is 274 m (900 ft.), resulting in a maximum of 4364 hm^3 (3,538,000 acre-feet) of maximum storage, by subtracting the designated space for flood control it is possible to determine the minimum surface elevation for flood control, which is at elevation 258.62 m (848.5 ft), with a storage capacity of 3439 hm^3 (2,788,000 acre-feet). Furthermore, according to the same manual, the main spillway capacities are 4,247 m^3/s (150,000 cfs) at reservoir elevation 263.19 m (863.5 ft.) and above, and 2,406 m^3/s (85,000 cfs) at the minimum surface elevation for flood control, 258.62 m. As stated above, the sill elevation of the flood control outlet is 248 m (813.6 ft.).

Based on the CDEC dataset, storage surpassed the minimum flood control elevation at 7:00 AM on February 15, peaked, then gradually declined, remaining above the minimum elevation until 0:00 AM on March 15th.

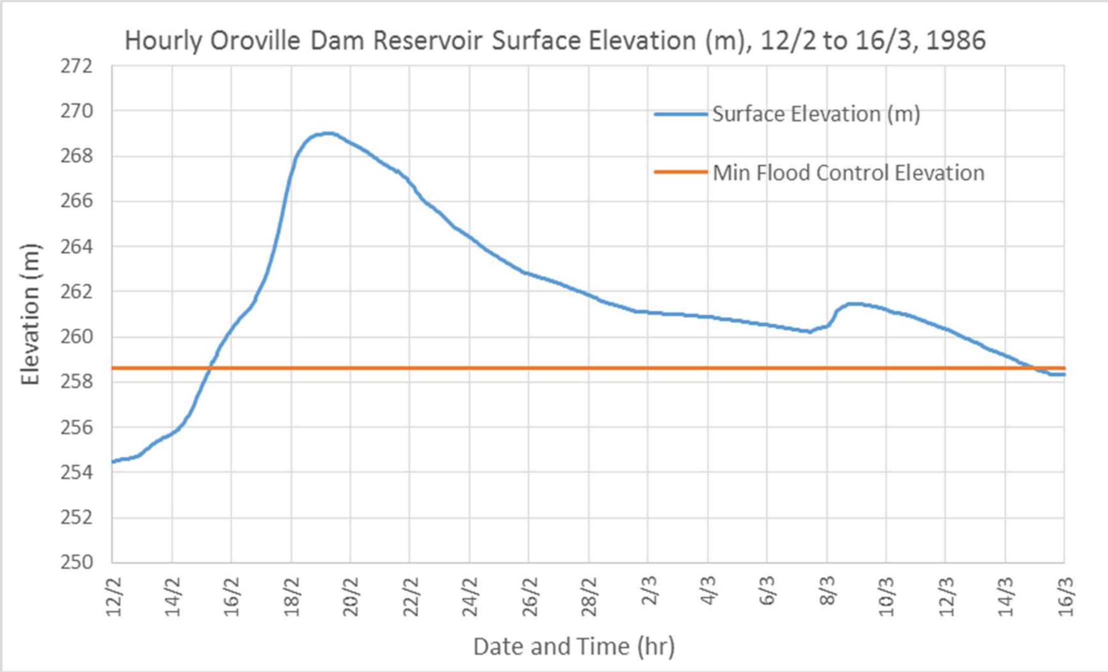


Figure 19. Hourly Oroville Dam Reservoir Elevation (m), February 12th to March 16th, 1986.

6.3 The 1997 Flood

Northern California was affected by a series of storms between December 26, 1996 and January 3, 1997. These were yet again caused by a "Pineapple Express" atmospheric river event, described in detail by (Kozlowski & Ekern, 2012).

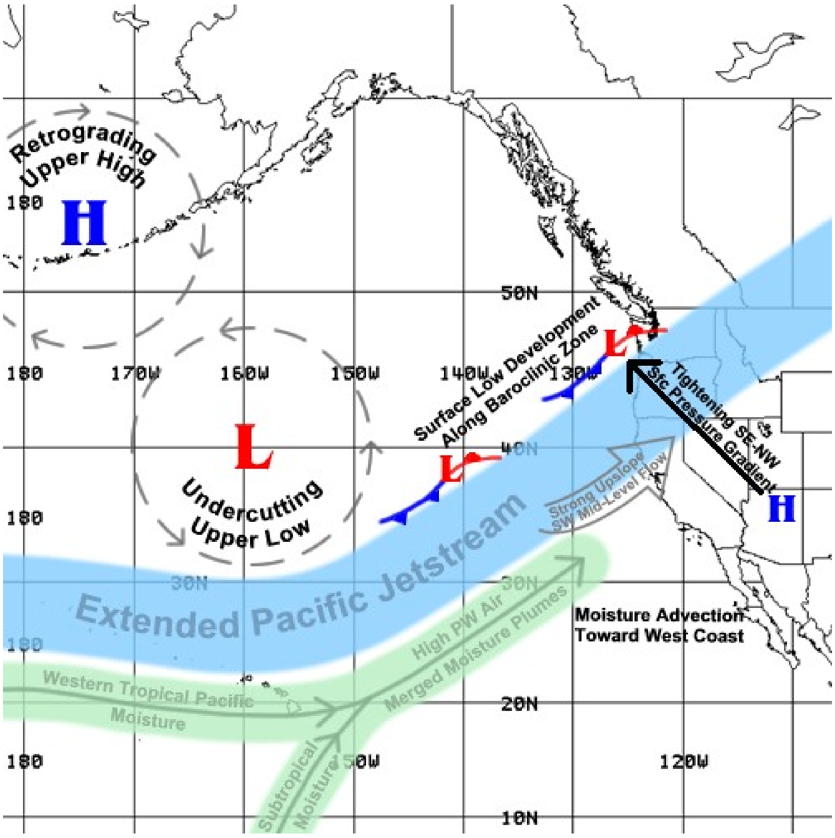


Image 9. The general weather pattern of the 1997 "Pineapple Express". Sources: CNRFC, NOAA, 2012.

In brief, this phenomenon began when the upper level ridge aligned along the United States west coast began to shift westward. This resulted in an influx of cooler air incoming from Canada. A deep upper level low undercut the ridge near 40ºN 160ºW, causing an extension of the Pacific jetstream over the Hawaiian Islands, towards southern Oregon and Northern California. This, combined with a development of low pressure surface areas offshore along the baroclinic zone resulted in a prolonged period of increased surface dewpoint temperatures, and an increase in south and southeastern winds, pushing a warm subtropical air mass towards the Central Valley.

This weather pattern brought warm, tropical storms from December 26, 1996 through January 3, 1997, with the most intense event occurring on January 1st. In addition, a previous cool winter storm occurring around December 21 left several feet of snow in higher elevations throughout northern California, which would contribute to the total streamflow later on when the tropical rainstorms struck the region.

Analysis of precipitation data from CDEC and NOAA (California Department of Water Resources, 2017); (Menne, et al., 2015) yield the following results for the time span of December 20, 1996 to January 4, 1997:

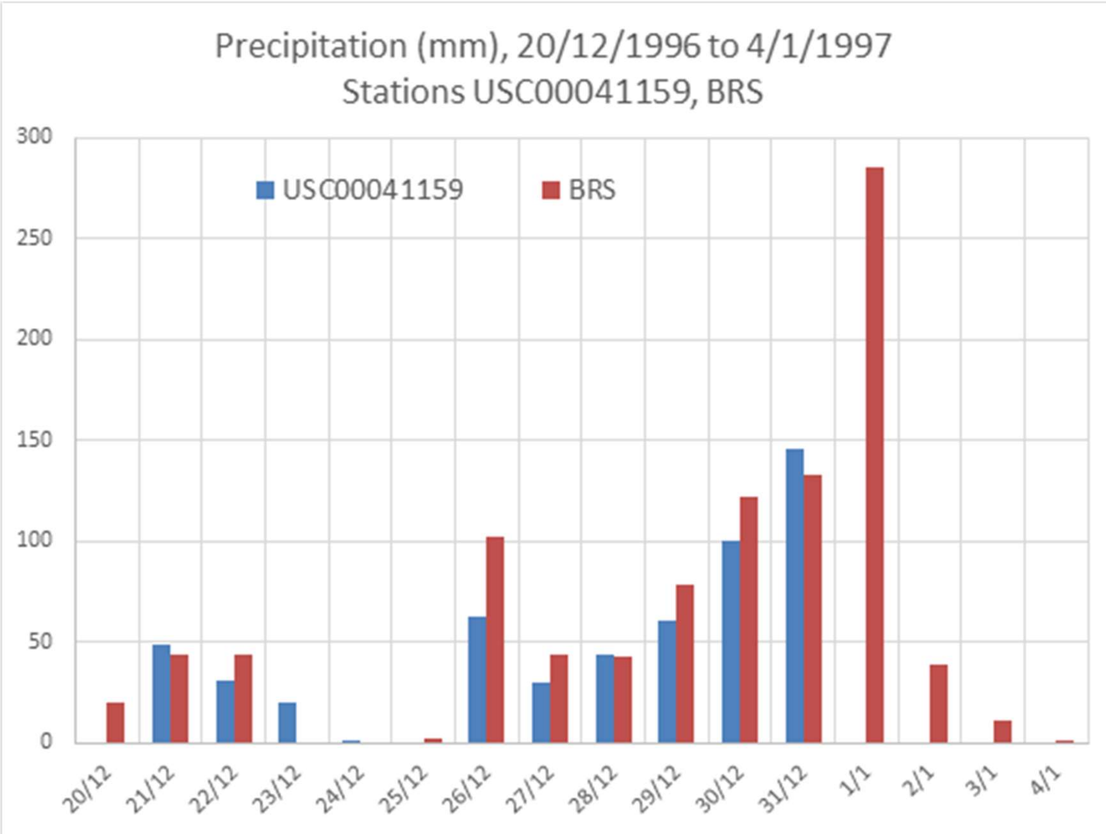


Figure 20. Daily total precipitation in mm, stations USC00041159 and BRS, from December 20, 1996 to January 4, 1997.

The above graph clearly shows two distinct precipitation events occurring in the Feather Basin: One 3-day storm from approximately December 20th to the 23rd, peaking at 48 mm (1.9 inches) (Station USC00041159), and at 43 mm (1.7 inches) (Station BRS), and one significantly larger storm. The latter is a 6-day event for station USC00041159, peaking at 146 mm (5.73 inches) and and 10-day event for station BRS, peaking at 285 mm (11.22 inches). Total precipitation during this two-week span was 541.1 mm (21.3 inches) for Station USC00041159 and 965 mm (38 inches) for station BRS. In addition, (Kozlowski & Ekern, 2012) contains a 9-day precpitiation total for the nearby "Bucks Lake" station from the 26th of December to the 1st of March, which is 1071 mm (42.16 inches). Overall, this storm is the most potent event to ever occur until now in the Feather Basin according to the available records.

From a hydrologic standpoint, this event produced record high flows due to the existence of high snow levels from the previous cool storm, which quickly melted from the extreme rainfall that occurred shortly thereafter. Despite its magnitude, however, it is the expected series of events that would develop into a large flood, and Oroville Dam was designed with this in mind (California Deparment of Water Resources, 1974); (US Army Corps of Engineers, 1970).

The California Data Exchange Center (California Department of Water Resources, 2017) contains daily inflow and outflow data during this event, which are plotted below, starting from December 26th up to January 16th.

An important note: for the outflow data specifically, the DWR states the following:

> *“Outflow from Oroville includes all releases from the Oroville Dam (i.e.: Hyatt, spillway, low flow outlet)”*

-California Depatment of Water Resources, Oroville Dam (ORO) Station Comments, 23/02/17

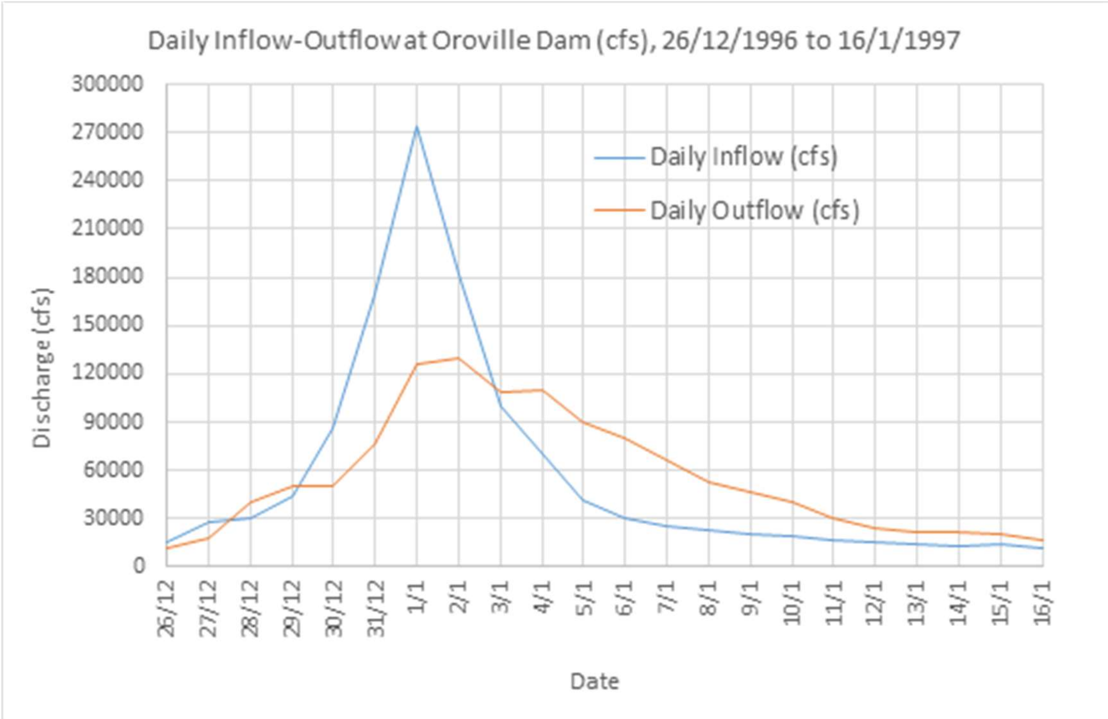


Figure 21. Daily Inflow-Outflow at Oroville Dam (cfs), from 26/12/1996 to 16/1/1997.

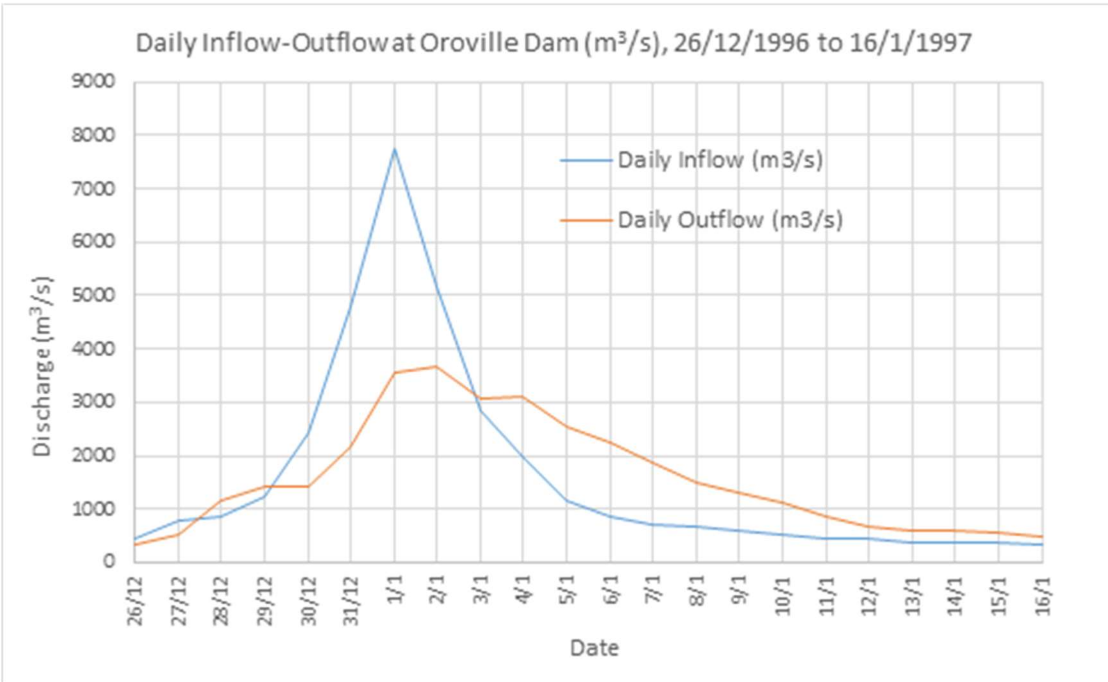


Figure 22. Daily Inflow-Outflow at Oroville Dam (m^3/s), from 26/12/1996 to 16/1/1997.

The overall daily peaks are 7,766 m^3/s (274,267 cfs) of inflow, occurring on the 1st of January 1997 and 3,660 m^3/s (129,256 cfs) of outflow, measured on the 2nd of January. For the same event, (Lamontagne, et al., 2012) measure 8,860 m^3/s (312,893 cfs) of 1-day maximum inflow. Furthermore, (Kozlowski & Ekern, 2012) have plotted a bi-hourly hydrograph of this flood, shown below.

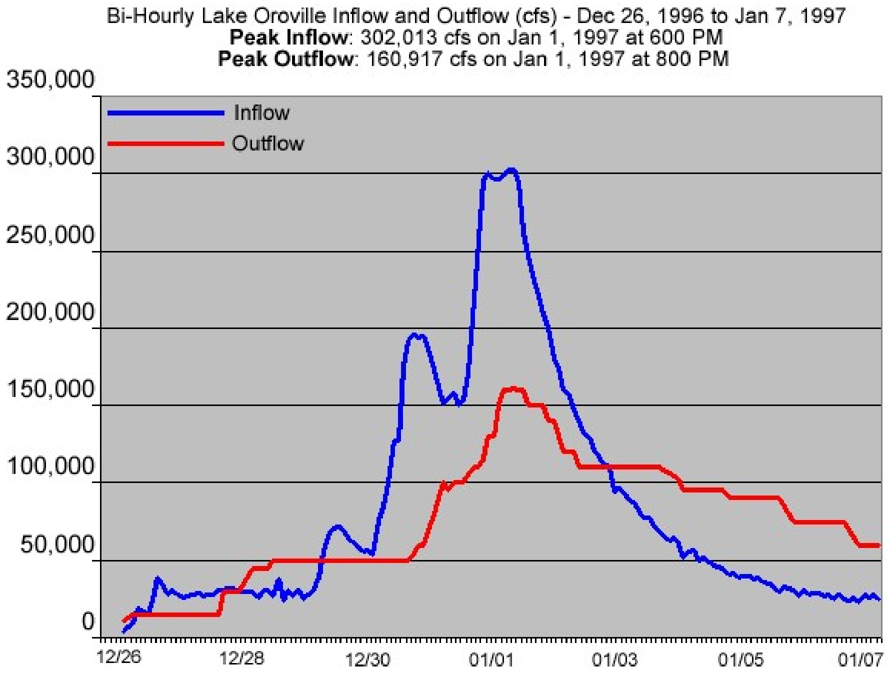


Figure 23. Bi-Hourly Lake Oroville inflow and outflow in cfs, from December 26, 1996 to January 7, 1997

Kozlowski and Ekern calculate bi-hourly peaks at Oroville Dam as follows: For inflow, 8,552 m^3/s (302,013 cfs), occurring on January 1st at 6:00 PM, and for outflow, 4,557 m^3/s (160,917 cfs). As is evident from this graph, spillway flows briefly exceeded the previous maximum of 150,000 cfs for approximately 6 hours. This is in accordance with the flood control manual (US Army Corps of Engineers, 1970) for high flows, and remained within 90% of inflow as designed.

Images of this record-high release are available below, courtesy of the Califorina Department of Water Resources.

Image 10. January 2, 1997. Side view of Oroville Dam's flood control outlet, as releases reach 4,531 m^3/s (160,000 cfs) for the first time in history. Sources: Norm Hughes, DWR.

Image 11. January 2, 1997. Top view of Oroville Dam's main spillway. Discharge is 4,531 m^3/s (160,000 cfs). Sources: Norm Hughes, DWR.

During the 1996-97 flood, the Oroville Dam reservoir elevation also reached record highs. Analysis of hourly Lake Oroville surface elevation data (California Department of Water Resources, 2017) yields the following graph:

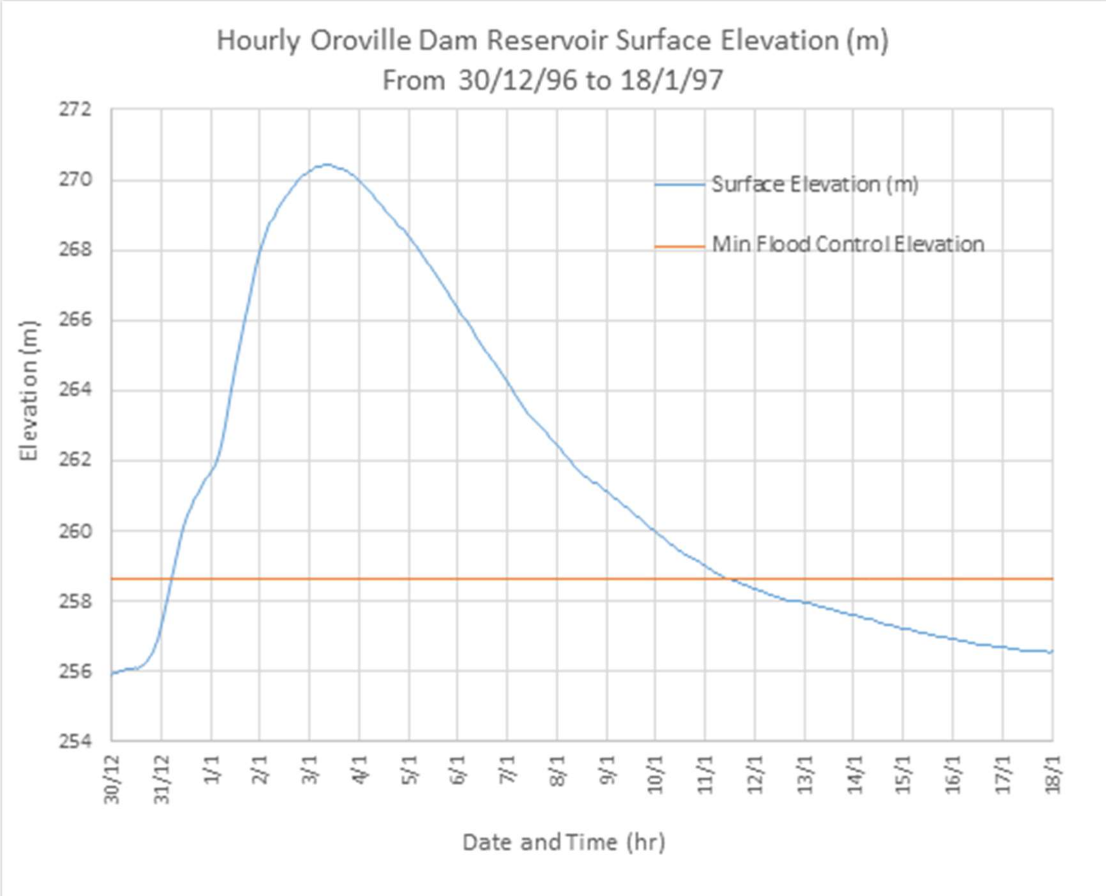


Figure 24. Hourly Oroville Dam Reservoir Surface Elevation in meters, from December 30th, 1996 to January 18th, 1997.

According to CDEC data, the surface elevation peak surpassed the minimum flood control elevation on December 31 at 5:00 AM, reached a peak of 270.42 m (887.19 m), then receeded below the minimum elevation again on January 11 at 12:00 PM. Maximum reservoir storage at peak elevation was 4,119 hm^3 (3,339,222 acre-feet).

6.4 The 2005 Oroville Dam Relicensing and Criticism

Starting from 2003, the California Department of Water Resources initiated a large-scale project with the purpose of renewing the United States Federal Energy Regulatory Commision (FERC) license to operate the hydroelectric facilities of Oroville and Thermalito Diversion Dams. This effort consisted of a number of various new scientific studies, as well as a collection of all previous ones related to the

Feather Basin, including but not limited to, several environmental impact reports, water quality certifications, and a detailed flood management study, (California Department of Water Resources, 2004). The latter contains a compilation of known flood control studies up to that year related to the Feather River. The most important ones pertaining to Oroville Dam consist of a water surface analysis of the river complete with floodplain maps, a forecast-based operation study for the dam, emergency action plans in case of severe accidents, and updates to the Probable Maximum Flood (PMF) for Lake Oroville, including a reconstruction of the previous 1986 and 1997 floods using the HEC-RAS software. The data from this report are the most recent available, and will be analyzed later in this thesis.

The DWR officially applied to the FERC for a new license on January 26, 2005. On February 1, 2007, the FERC officially authorized the DWR to continue operation of the Oroville Facilities until January 31, 2008. Until then, this license has yet to be renewed as of today (Johnson, 2017).

However, the relicensing process was not met with universal approval. On October 17, 2005, three independent parties, the Friends of the River (FOR), the South Yuba River Citizens League (SYRCL), and the Sierra Club filed a motion to intervene in the Oroville relicensing (Stork, et al., 2017), citing that under current conditions, the emergency spillway was not prepared for the expected design discharges stated in the 1970 flood control manual. Detailed comments can be found in a subsequent letter, sent to the Federal Energy Regulatory Commission by the same group on Decemmber 18, 2006. This letter (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006) contains their comments on the dEIS (Draft Environmental Impact Statement, FERC/DEIS-0202D). FOR and others state that the dEIS does not include construction plans necessary to conduct surcharge operations of regulated flows consistent with the existing 1970 flood control manual. A direct quote from this correspondence states that:

> *"The dEIS is silent on how the* ***existing*** *structural deficiencies of the Oroville Dam facilities that affect the willingness of its operators to conduct operations required by* ***existing*** *Corps regulations will be addressed."*

-Friends of the River; Sierra Club; South Yuba River Citizens League, Comments on the dEIS (2006).

According to (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006), the absence of armoring on the emergency spillway means that any flood discharges may cause significant erosion and damage downstream project lands and facilities, and mentions that this design flaw is inconsistent with current FERC

"Engineering Guidelines", which did not exist at the time of the auxiliary spillway's construction.

Furthermore, in the same letter FOR, et al., state that none of the project safety facilities proposed by interest groups intended for the protection of downstream communities were included in the dEIS, and comment that this shows a lack of responsibility from the DWR's side. They add that this decision is likely related to the fact that emergency use of the auxiliary spillway would likely not result in failure of the main dam crest, but there is no publicly available official document to confirm this.

Next, this correspondence includes a segment from the group's October intervention, which noted that the 1970 flood control manual (US Army Corps of Engineers, 1970), requires the use of the auxiliary spillway for regulated operational releases. However, when the dam was initially licensed, this untested ogee weir was called an "emergency" spillway instead. According to FOR, et al., under the current manual, it would seem better to characterize the emergency spillway for the first 3 m (10 feet) of flow as an auxiliary spillway, noting that precision in language is important, as use of an "emergency" spillway would entail more expected damage to downstream facilities than an "auxiliary" structure.

Finally, the letter concludes with a comment aimed at the DWR's analysis of the 1997 flood:

> *"Deciding the true probability of the 1997 event is at best an exercise in theological speculation. Regardless, it occurred less than ten years ago, and the event was smaller than the Corps design flood for the Feather River at Oroville."*

-Friends of the River; Sierra Club; South Yuba River Citizens League, Comments on the dEIS, 2006

6.5 Summary of Recent Spillway Inspections

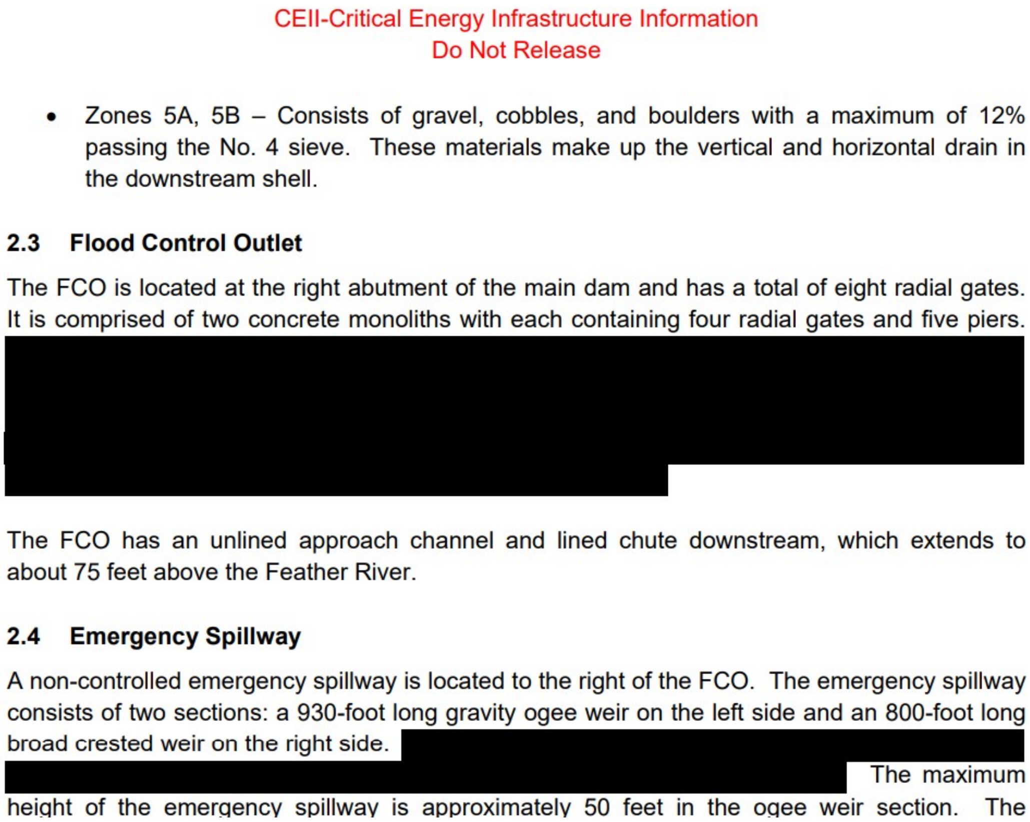
This chapter contains important findings from official inspections of Oroville's main and auxiliary spillways. These inspections were conducted by the California Department of Water Resources Dam Safety Division, and the Federal Energy Regulatory Commission (FERC).

6.5.1 2009 Potential Failure Mode Analysis (PFMA), FERC

The 2009 Potential Failure Mode Analysis Summary (Federal Energy Regulatory Commission, 2009) is one of several reports produced by an independent board of consultants, which are then submitted to FERC for review. The purpose of the PFMA summaries is to examine possible failure modes for Oroville Dam and assess the safety of its facilities. FERC conducts one independent investigation every five years.

This report contained information relative to the emergency spillway, noting the existence of heavy vegetation below the emergency spillway crest. Several trees were directly in its channel, which in the event of a severe storm that required its operation, would possibly be uprooted and accumulate downstream as debris.

It is important to note that several passages of this report and all following documents supplied by FERC have been heavily redacted, because they contain "Critical Energy/Electric Infrastructure Information" (CEII). An example of redacted passages can be seen below.


CEII-Critical Energy Infrastructure Information
Do Not Release

- Zones 5A, 5B – Consists of gravel, cobbles, and boulders with a maximum of 12% passing the No. 4 sieve. These materials make up the vertical and horizontal drain in the downstream shell.

2.3 Flood Control Outlet

The FCO is located at the right abutment of the main dam and has a total of eight radial gates. It is comprised of two concrete monoliths with each containing four radial gates and five piers. [redacted]

The FCO has an unlined approach channel and lined chute downstream, which extends to about 75 feet above the Feather River.

2.4 Emergency Spillway

A non-controlled emergency spillway is located to the right of the FCO. 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. [redacted] The maximum height of the emergency spillway is approximately 50 feet in the ogee weir section. The

Image 12. Redacted passages containing CEII information on the 2014 PFMA FERC report.

CEII information is defined by the FERC as follows (Federal Energy Regulatory Commission, 2016):

> "*Specific engineering, vulnerability, or detailed design information about proposed or existing critical infrastructure (physical or virtual) that:*
>
> 1. *Relates details about the production, generation, transmission, or distribution of energy;*
> 2. *Could be useful to a person planning an attack on critical infrastructure;*
> 3. *Is exempt from mandatory disclosure under the Freedom of Information Act; and*
> 4. *Gives strategic information beyond the location of the critical infrastructure."*

-CEII definition, as given by FERC

An option is given to file a request to obtain the original versions of redacted CEII documents, but this requires signing a non-disclosure agreement, which would bar the reveal of any non-public data in this thesis. While this measure can be appreciated as an extreme precaution with the safety of the population in mind, in order to avoid possible terrorist attacks on critical engineering structures in the United States, the concept of withholding data from the public is wholly inconsistent with the basic foundations of science and engineering, which are based on peer review. At the very least, some explanatory data should be provided next to every redaction, to give clues as to what specific element is being redacted and why. Regardless, the available public documents still do contain important data.

6.5.2 2014 Potential Failure Mode Analysis (PFMA), FERC

The very next FERC report (Federal Energy Regulatory Commission, 2014) confirms that the DWR has been performing minor repairs to the main spillway chute as recently as 2009. A brief Internet search revealed photos of this chute maintenance. It appears to consist of filling in cracks in the spillway chute with additional concrete.

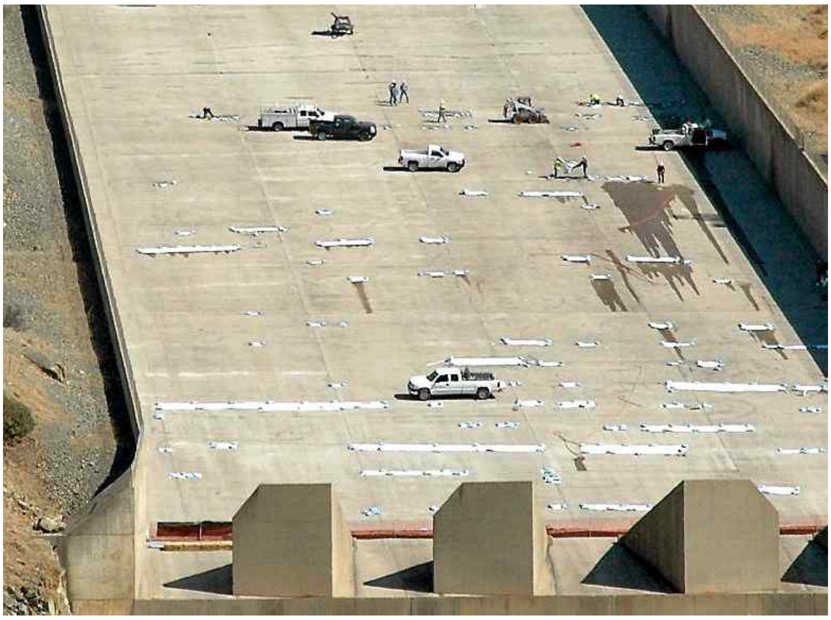


Image 13. Maintenance of the Oroville Dam main spillway chute. Photo taken in 2009. Source: Barbara Arrigoni.

A second image, taken in 2013, shows additional chute maintenance occurring further up along the chute axis. Further data was not available from the FERC report, but sources indicate that this is a routine maintenance practice that has occurred several times before. (Olenyn, 2017)

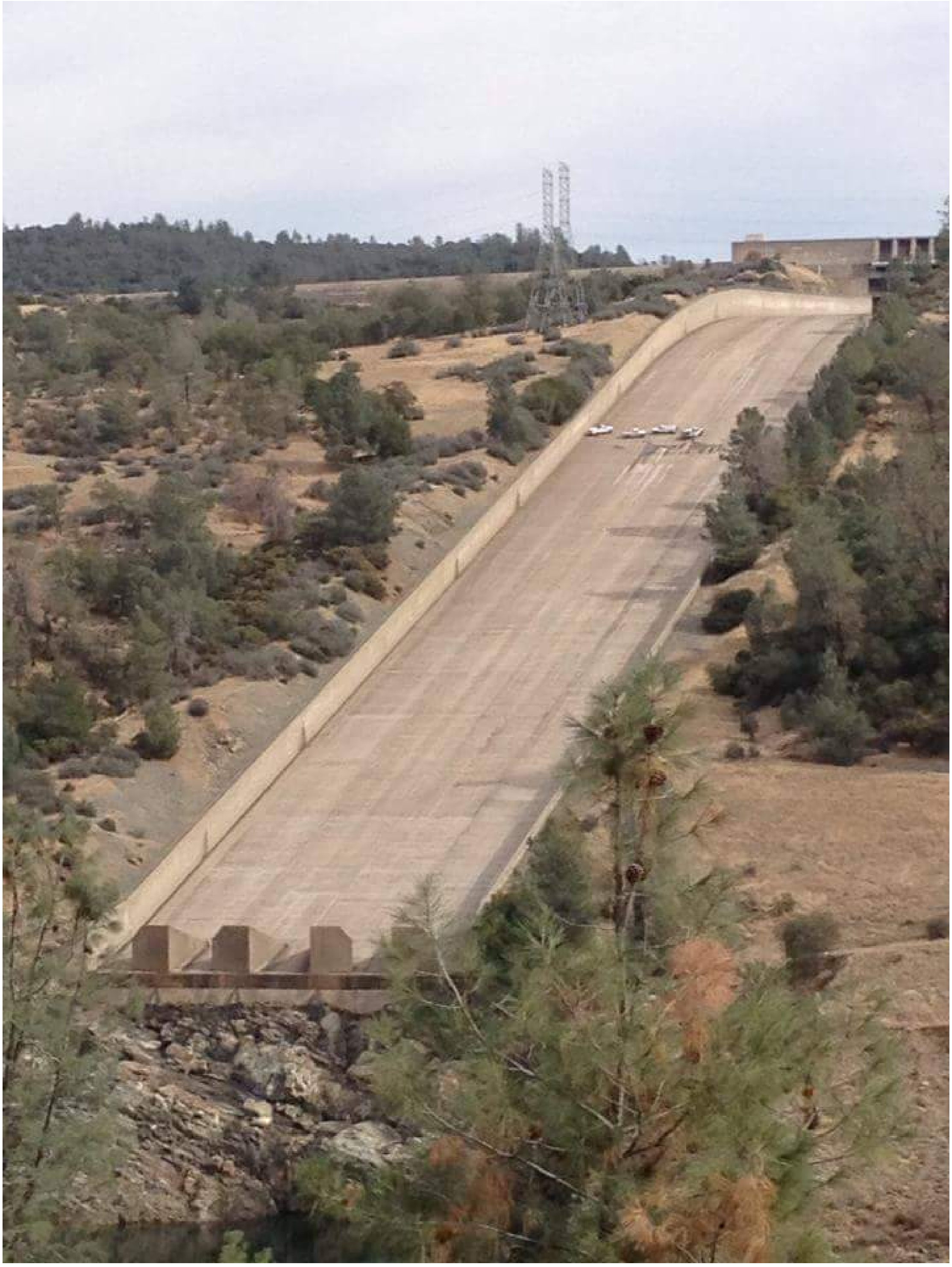


Image 14. 2013 image showing repairs being made on the Oroville main spillway chute. Source: Unknown, possibly DWR.

6.5.3 January 8th, 2013 Division of Dam Safety Inspection

This inspection (Division of Safety of Dams, CA DWR, 2013), conducted by the DWR's Division of Dam Safety, contained the following findings for the Oroville spillways:

> *"The emergency spillway appeared to be stable and well aligned. The concrete comprising the emergency weir and the Flood Control Outlet (FCO) headworks appeared to be in satisfactory condition. No signs of instability were observed at the FCO."*

Also mentioned is a previously applied segment of orange monitoring paint, used to detect concrete spalling at the spillway bridge and on an existing small diagonal crack on the left bridge abutment. However, this paint was found to be undisturbed, and largely remained so in later reports.

6.5.4 July 15th, 2013 Division of Dam Safety Inspection

The second report in 2013 (Division of Safety of Dams, CA DWR, 2013) contains the following information regarding the Oroville spillways:

> *"The Flood Control Outlet structure (FCO) was viewed from the service deck (top), the radial gate hoist deck, the roadway bridge, and the trunnion inspection deck. The discharge chute was inaccessible due to the seal leakage flow and our concern for worker safety. The chute was observed from the FCO decks and from the opposite side of the river. [...] The emergency spillway weir and downstream apron appeared to be well aligned. The FCO discharge chute walls appeared to be stable and in satisfactory condition."*

The aforementioned seal leakage referred to the FCO gate seals, which were gradually being replaced at the time.

6.5.5 August 3rd, 2015 Division of Dam Safety Inspection

The 2015 safety inspection (Division of Safety of Dams, CA DWR, 2015) revealed the following:

> *"The approach channel was clear and the security barrier was beached. [...] The concrete training walls remain stable appearing and in good condition.*

The FCO appeared to be in satisfactory condition. […] The full length of the FCO discharge chute was inspected. Conditions appear to be normal. The concrete repairs along the chute floor remain sound. The walls were well aligned and appeared to be stable. […] 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. Conditions along the emergency spillway weir were unchanged from recent inspections. The structure was stable appearing, and the concrete remains sound."

The aforementioned "lone tree" is shown in the image below.

Image 15. August 3, 2015. A "lone tree" along the left wall of the Oroville Dam main spillway. Source: Division of Dam Safety, DWR.

6.5.6 August 22nd, 2016 Division of Dam Safety Inspection

This report (Division of Safety of Dams, CA DWR, 2016) is the latest available. It contains the following information related to the Oroville Dam spillways:

> *"The approach channel was fully exposed and clear, and the security barrier was beached. The concrete approach walls remain stable appeaing and in good condition. The FCO appreaded to be in satisfactory condition. […] The FCO discharge chute was inspected from the top of the outlet structure, the trunnion deck, and the road across the river channel. Conditions appeared to be normal. The chute walls were well aligned and appeared to be stable. […] Vegetation has been removed from behind the [left] wall. […] Conditions along the emergency spillway weir were unchanged from recent inspections."*

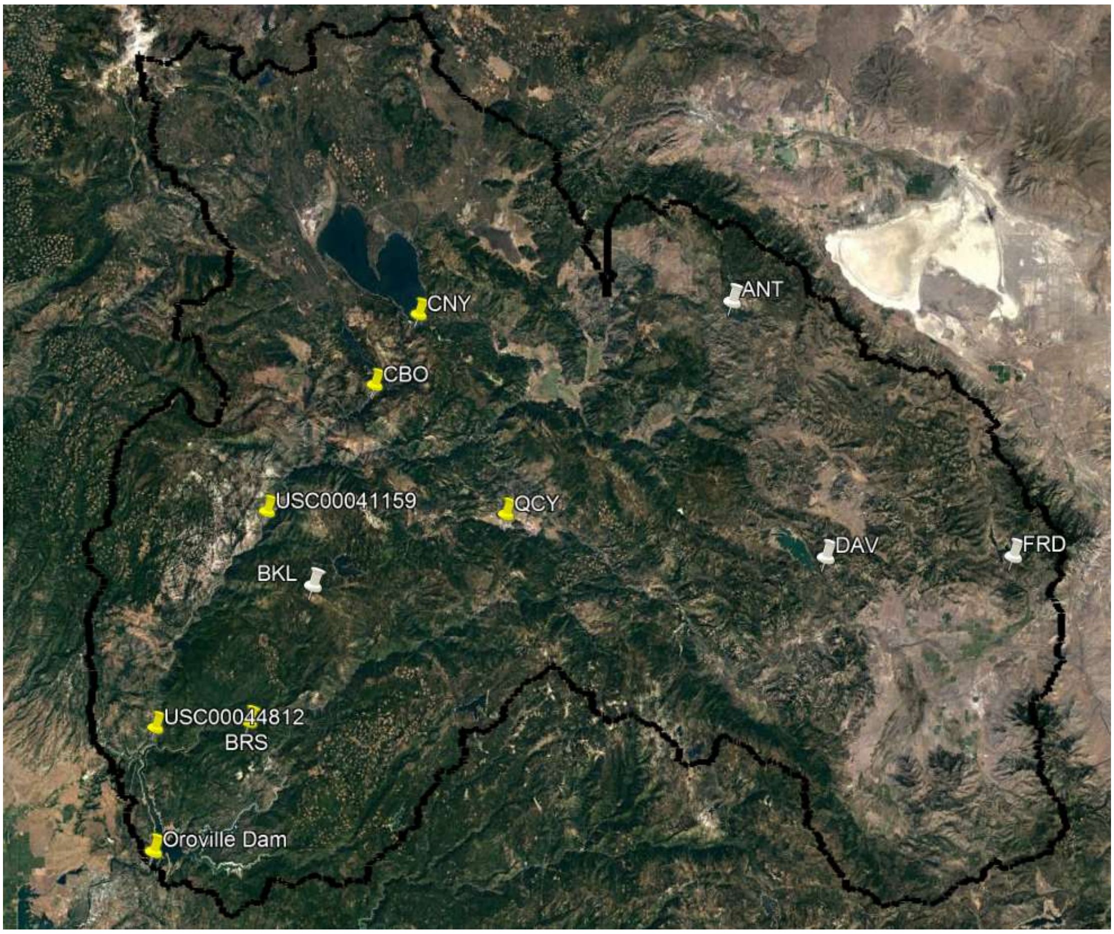
7. The 2017 Spillway Incident

7.1 The January 2017 Storm

Of the so far analyzed precipitation measurement stations, none have records of the year 2017, so a new batch of CDEC stations (California Department of Water Resources, 2017) are analyzed. Important data and a combined map of these new stations together with the previously examined ones can be found below. The analyzed record for all stations starts on January 1st 1987, and ends at October 1st, 2017, with the exception of Bucks Lake, where the record starts on October 1st, 1996 and ends on October 1st, 2017.

Table 16. New analyzed precipitation station names and locations.

Station Name	Station ID	Latitude	Longitude	Data Source
Bucks Lake	BKL	39.850	-121.242	CA DWR/ O & M
Antelope Lake	ANT	40.180	-120.607	CA DWR/ O & M
Frenchman Dam	FRD	39.883	-120.183	CA DWR/ O & M
Lake Davis	DAV	39.883	-120.467	CA DWR/ O & M



Map 7. Combined map of all analyzed precipitation stations. New ones are color-coded white, and the black outline is the border of the Feather River watershed. Source: Google Earth (2017).

An issue that arises with these new stations is the relatively low amount of recorded years (just above 30 years of data). This hinders the long-term prediction capabilities of a scientific analysis, as there are no data for known historic floods, such as the ones that occurred in 1907, 1964, and 1986. However, they are suitable for analysis of the 2017 storms.

During the first few days of January 2017, two small rain storms occurred just over Lake Oroville. Brush Creek station (BRS) from the CDEC database (California Department of Water Resources, 2017) reports the following data:

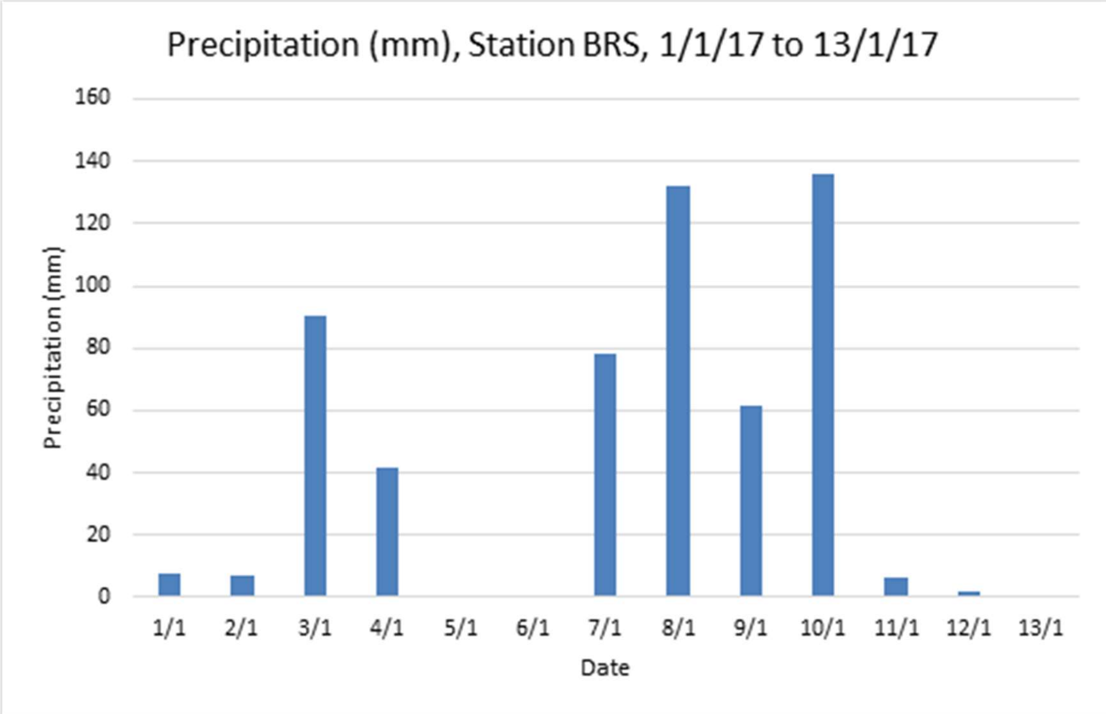


Figure 25. Daily precipitation (mm), Brush Creek (BRS) station, January 1, 2017 to January 13.

The first rain storm was short, lasting only 4 days, peaking at 90 mm (3.56 inches) on January 3, and the second was a stronger 6-day event, peaking at 136 mm (5.34 inches) on January 10. These rain storms quickly led into a large increase of inflows into Lake Oroville, shown below together with corresponding outflows, on an hourly scale.

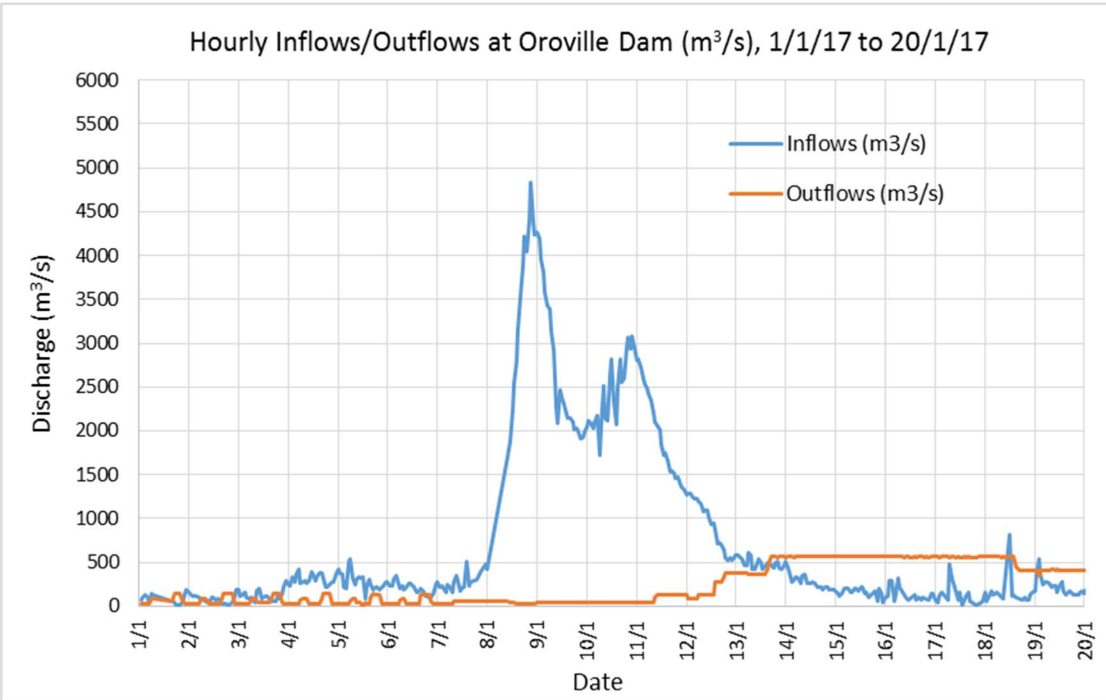


Figure 26. Hourly Inflows and Outflows at Oroville Dam in m³/s, from January 1, 2017 to January 20.

Two inflow peaks occurred according to the graph, the primary one was 4,839 m^3/s (170,887 cfs) on January 8 at 21:00 PM, and a secondary peak of 3,079 m^3/s, occurring on January 10 at 22:00 PM. These inflows are definitely significant, yet expected during a typical wet season. However, outflows from Lake Oroville at the same time were very low, almost zero. This resulted in a sharp water storage increase in Lake Oroville, as well as a significant rise in its surface elevation, plotted below next to the designated minimum flood control elevation of 258.62 m (848.5 ft.) as mentioned in the 1970 manual (US Army Corps of Engineers, 1970).

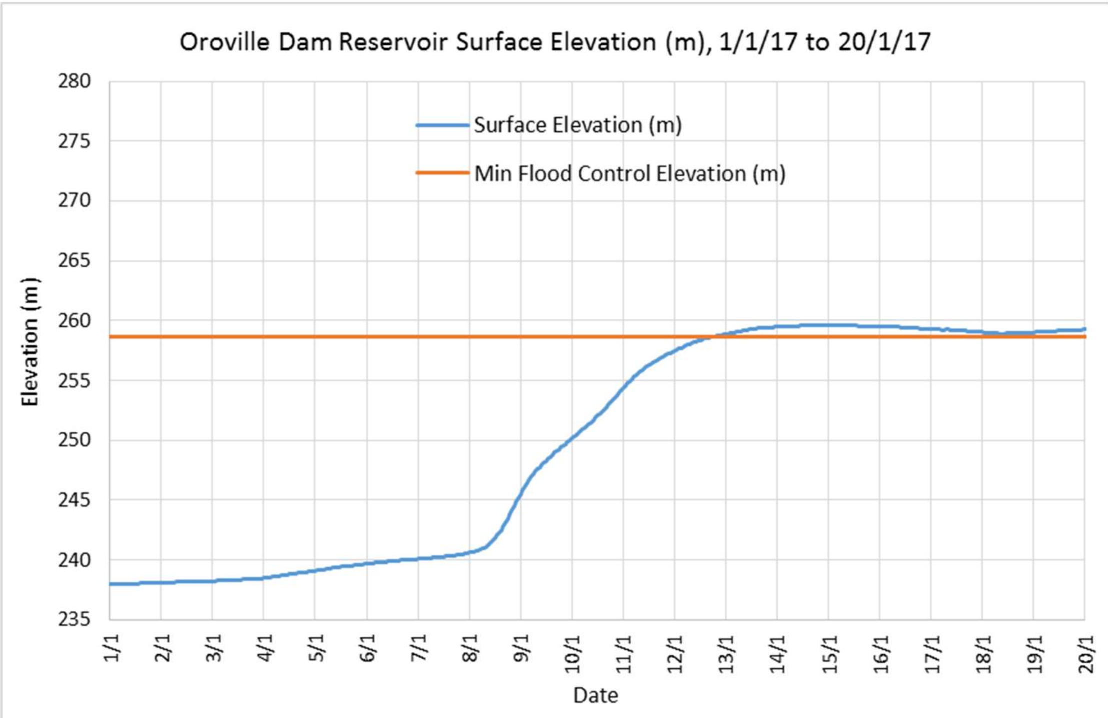


Figure 27. Lake Oroville Surface elevation in meters, from January 1, 2017 to January 20.

As is made clear from the graph, Lake Oroville's surface elevation initially exceeded the flood control minimum on January 12, 2017 at 17:00 PM. Around that time, outflows from Oroville Dam's main spillway were increased, to compensate for this fact and return the surface elevation to below the minimum. This attempt continued throughout the rest of January and is visualized in the following graphs.

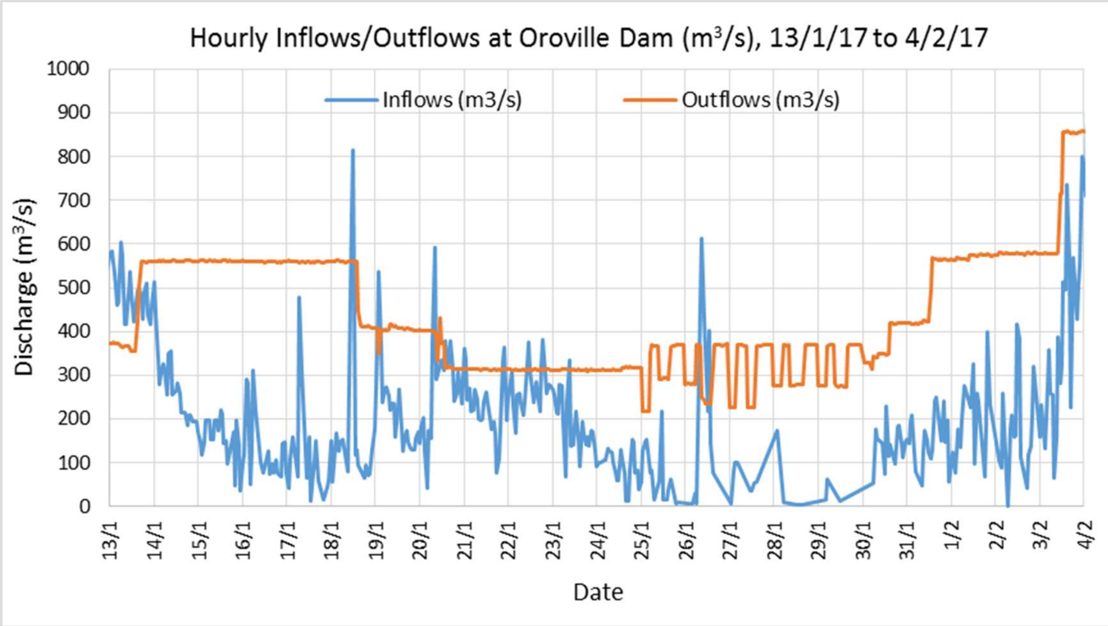


Figure 28. Hourly Inflows and Outflows at Oroville Dam in m^3/s, from January 13, 2017 to February 4.

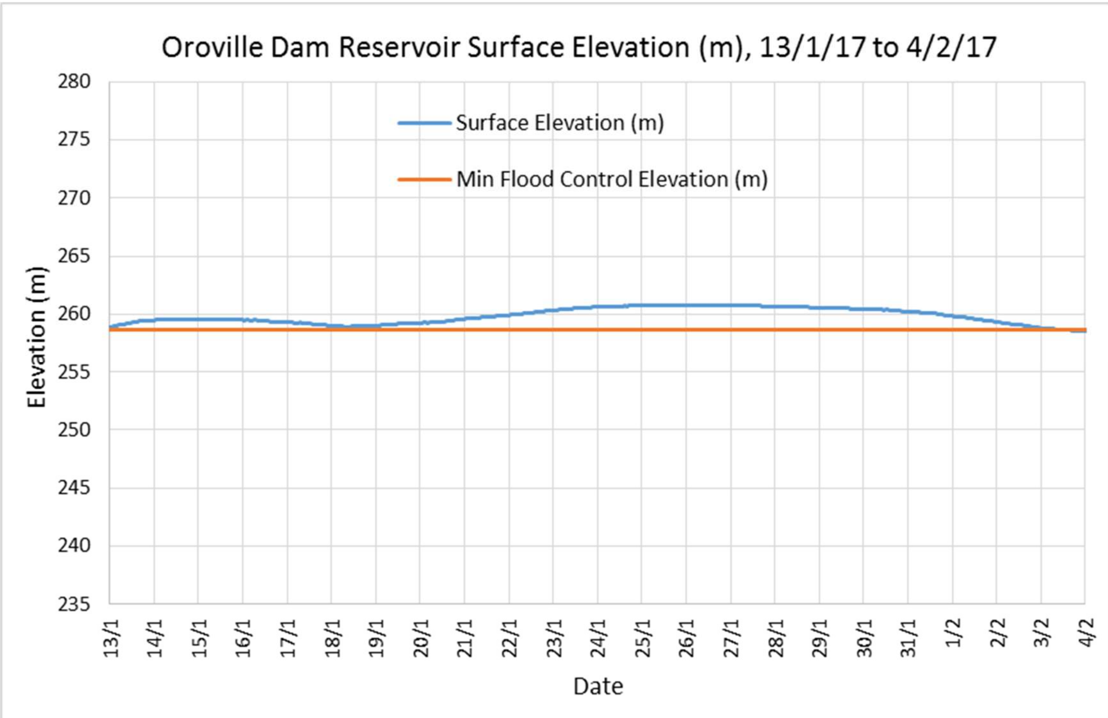


Figure 29. Lake Oroville Surface elevation in meters, from January 13, 2017 to February 4.

Overall, the Oroville Dam operator was able to return the surface level to below the flood control limit on February 3, 2017 at 17:00 PM, just in time for an upcoming February rain storm.

7.2 The February 2017 Storm

According to CDEC, a rain storm over the Feather Basin began on February 2, 2017, and ended around February 11. Data of this event are plotted below.

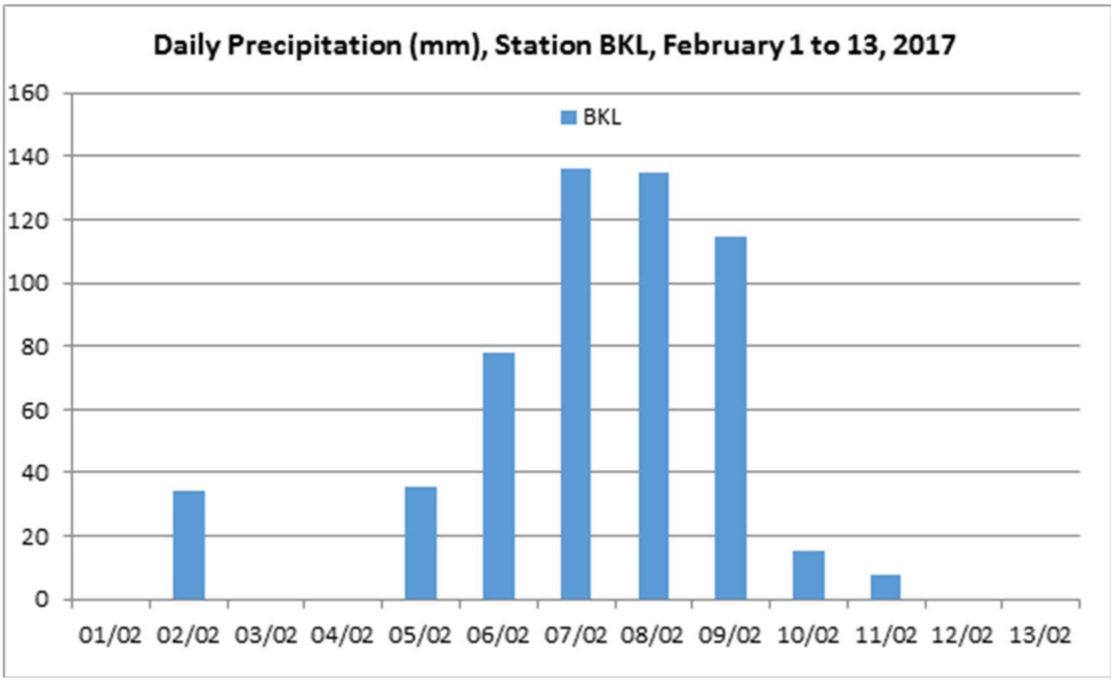


Figure 30. Daily Precipitation (mm), Bucks Lake (BKL) station, February 1 to 13, 2017.

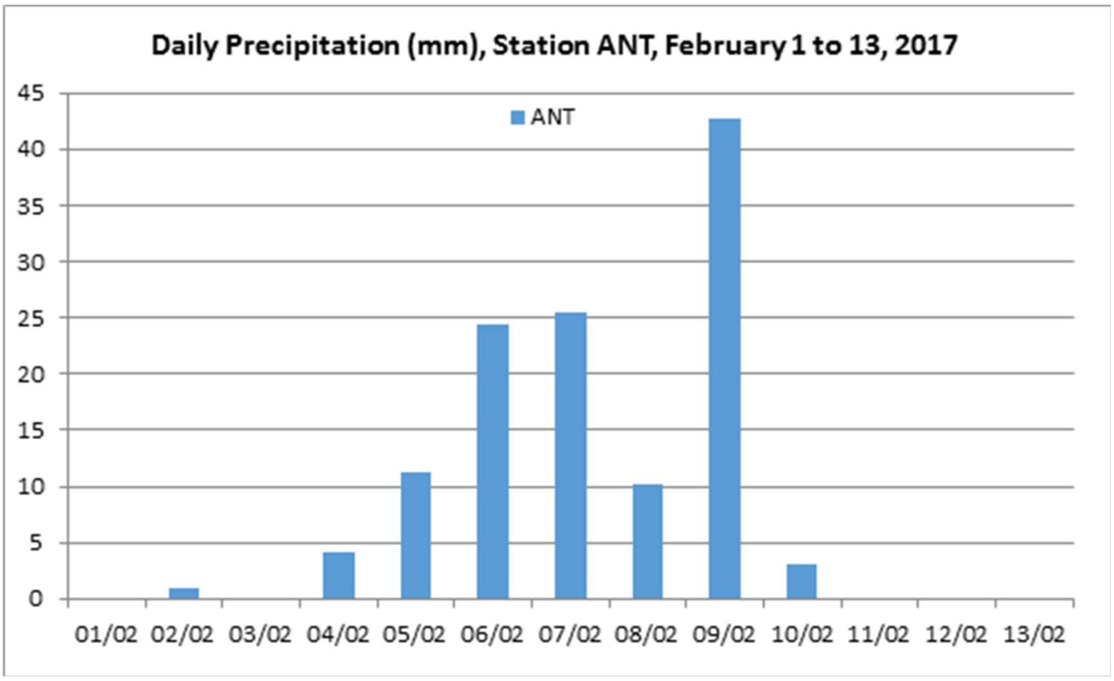


Figure 31. Daily Precipitation (mm), Antelope Lake (ANT) station, February 1 to 13, 2017.

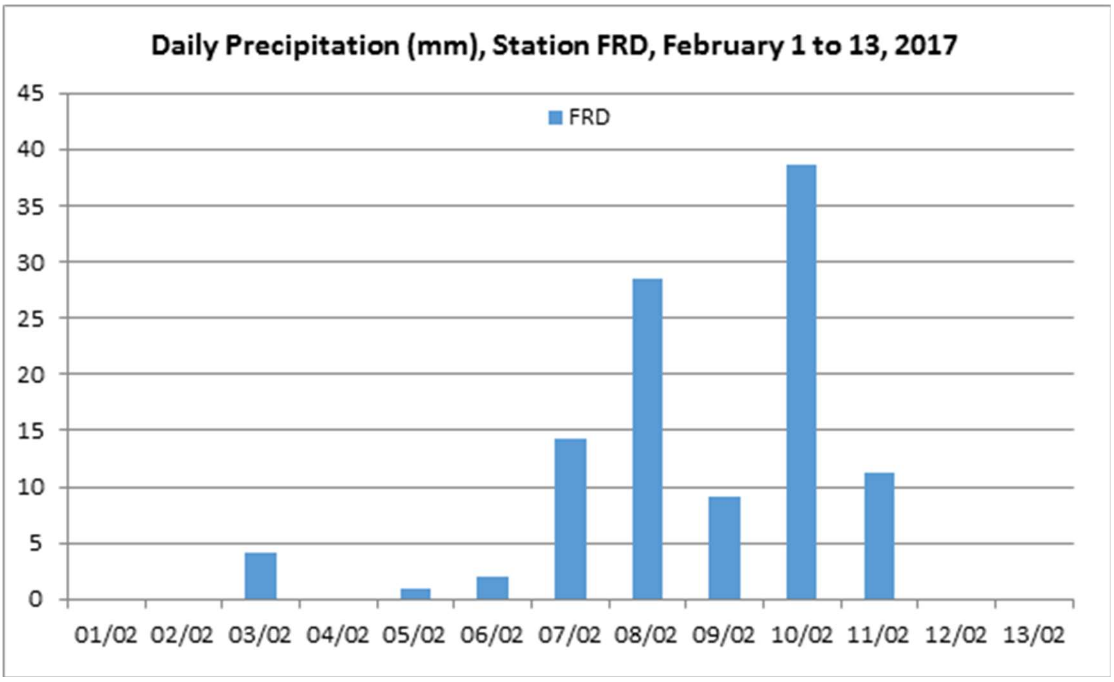


Figure 32. Daily Precipitation (mm), Frenchman Dam (FRD) station, February 1 to 13, 2017.

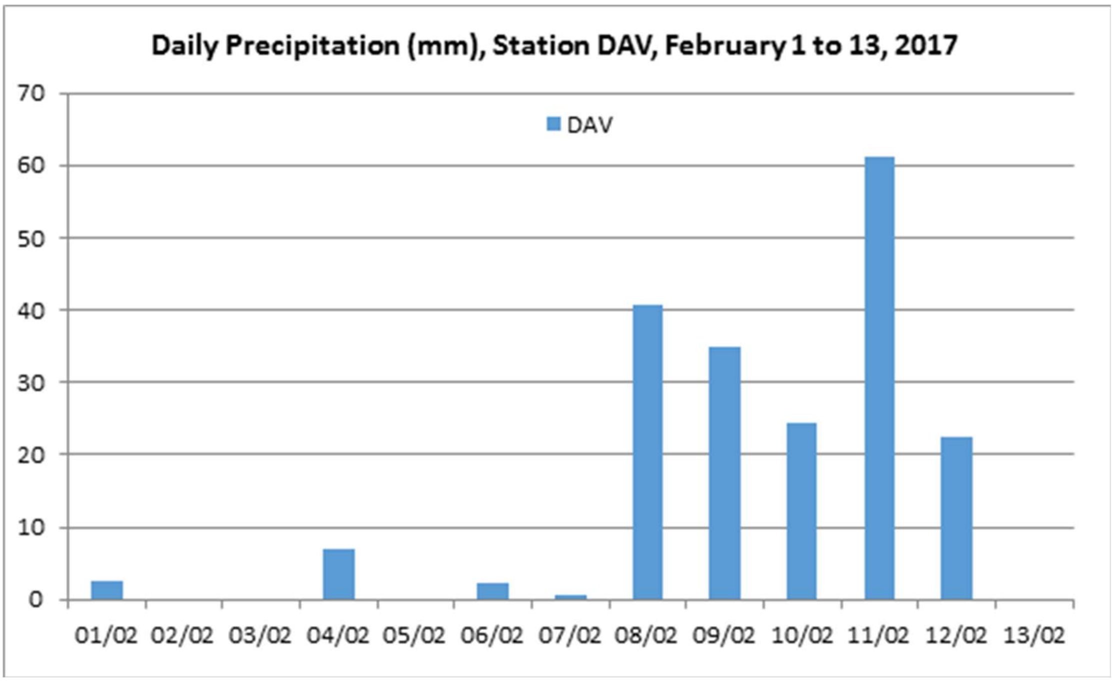


Figure 33. Daily Precipitation (mm), Davis Lake (DAV) station, February 1 to 13, 2017.

From the above analysis, a pattern emerges for the early February 2017 storm. This event appears to reach its peak just as it passes above Lake Oroville, with the highest precipitation value occurring at station BKL on the 7th of February, measuring 136 mm (5.36 inches) of rain. The storm then moved eastward, resulting in later peaks for the following stations. Moving from west to east, station ANT peaks at 43 mm (1.68 inches) on February 9, then station DAV peaks at 61 mm (2.41 inches) on February 11, and finally station FRD peaks at 39 mm (1.52 inches) on February 10. These reported amounts of precipitation are significantly lower than those of preceeding previous record floods. However, the fact that the rain storm seemed to peak near Lake Oroville should result in a brief high inflow peak.

7.3 February 2017 Inflows

The CDEC database contains hourly inflow and outflow data for the February storm. Inflow for the whole month of February 2017 is plotted in the graph below.

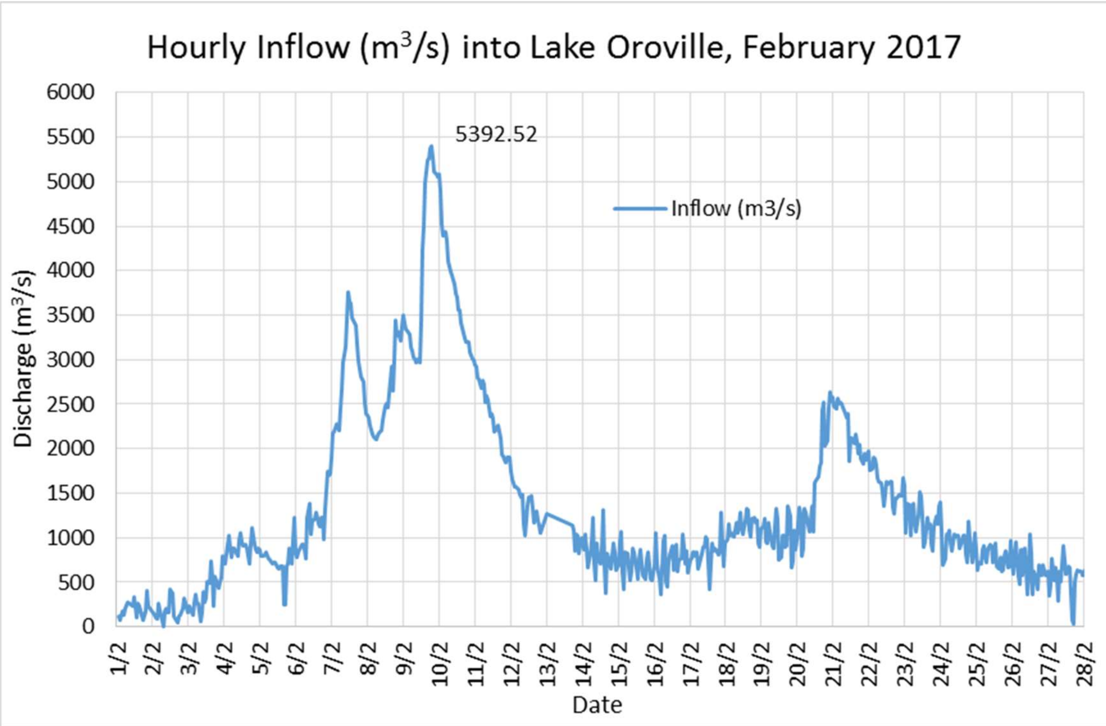


Figure 34. Hourly inflow into Lake Oroville in m^3/s, for the month of February 2017.

Overall, the early February rain storms seem to have resulted in subsequent inflows with peaks occurring shortly after peaks in the corresponding upstream precipitation measurement stations. The largest measurement occurred on February 9 at 19:00 PM, and was 5,392 m^3/s (190,435 cfs). This value is significantly lower than the highest recorded floods to ever occur in the Feather Basin, including the 1986 and 1997 floods. Instead, it more closely resembles the 1955 and 1964 floods in scale. Under normal circumstances, Oroville Dam should have been able to deal with this event without trouble.

7.4 February 7: Main Spillway Failure

On February 6 at approximately 13:00 PM, outflows from Lake Oroville were raised in order to prepare for incoming inflows to 1,500 m^3/s (54,000 cfs) . However, the next day, February 7, at approximately 10:00 AM, workers at the Oroville Dam site noticed a discoloration in the water flowing through the main spillway. Images of the spillway at that specific moment are not available, but an image taken later is a good approximation.

Image 16. February 8th, 2017. Discoloration of the flow along the Oroville Dam main spillway. Source: Kelly M. Grow, DWR.

Outflow from the main spillway was immediately halted, in order to detect the source of this discoloration, revealing a large hole in the main spillway chute.

Image 17. February 7th, 2017. Front view of the initial main spillway chute damage. Source: Kelly M. Grow, DWR.

At this point, the main spillway is already severely damaged, and any discharges at that point would rapidly amplify this erosion and move entire parts of the concrete chute and walls downstream. However, Lake Oroville's surface elevation is already past the flood control minimum, and inflows from the February rain storm are imminent.

A second image shows workers inspecting the newly damaged spillway. This helps appreciate the scale of the damage.

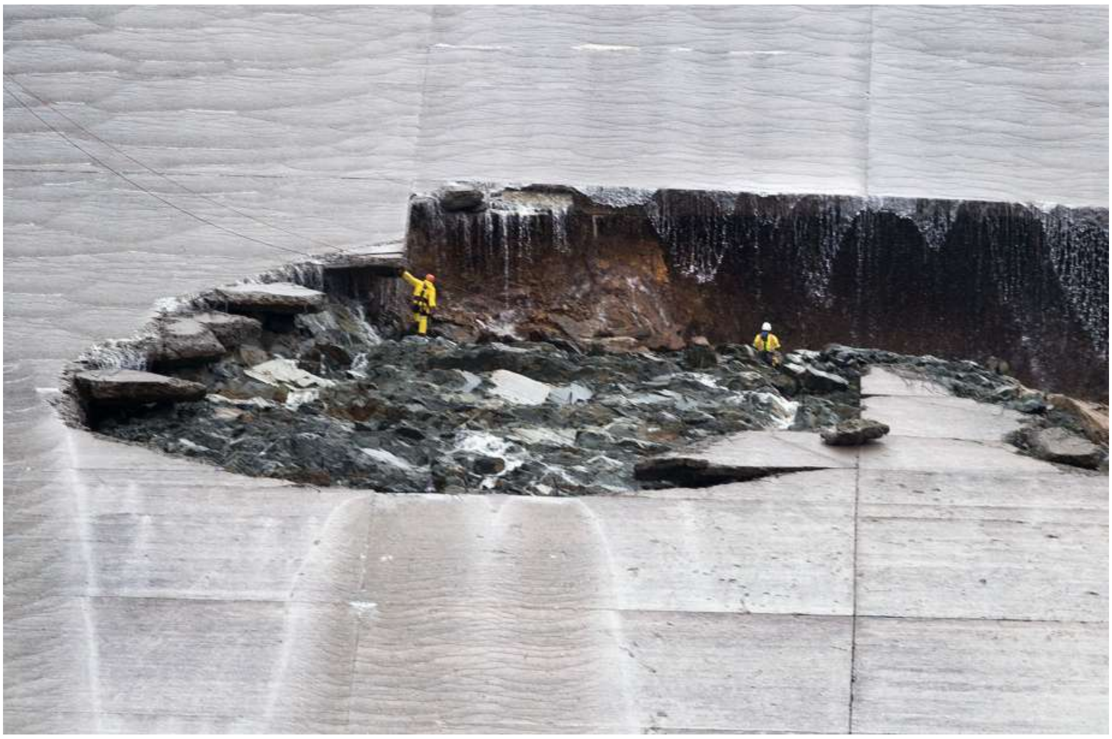


Image 18. February 7, 2017. Workers examining the ruptured Oroville Dam main spillway. Source: DWR.

7.5 February 8-10: Testing the Main Spillway

After brief consultation with various dam safety agencies, the operators decided to release test flows into the main spillway and monitor the damage. These small flows ranged hourly from around 300 m^3/s to 900 m^3/s (10,000 to 30,000 cfs) over the course of February 8th. On the very next day, February 9, the hole in the main spillway had increased in size, seen below compared to the initial February 7 picture.

Image 19. Comparison of the February 7 main spillway hole (left) to the damage on February 9 (right). The erosion appears to be moving uphill. Sources: Kelly M. Grow, DWR and the Metabunk.org forum.

A worrying aspect of the spillway damage is that it was moving uphill. This is a typical sign of a failure known as headcutting (or undercutting), which is what happens when water flowing across a hard surface, falls onto a softer surface below. A simple illustration below explains this concept.

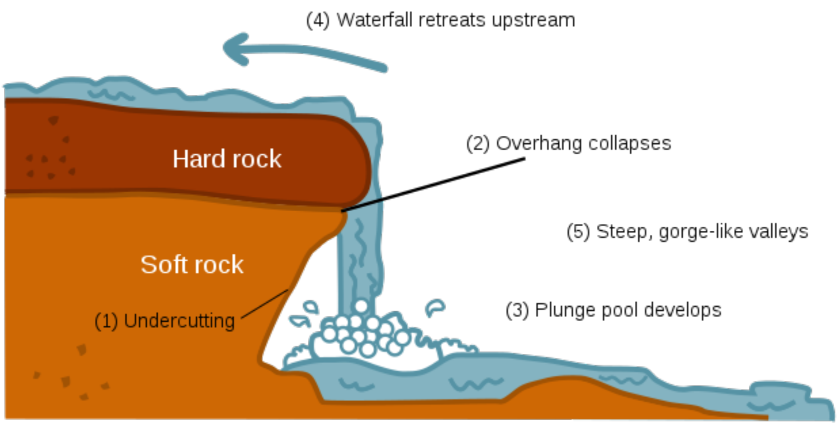


Figure 35. A typical example of undercutting failure. Source: Cradel, Wikimedia Commons (2009).

As in the above example, splashback from the newly created waterfall at the center of the main spillway caused it to erode in an upstream direction.

With the ever increasing inflows dangerously raising the reservoir surface level, which is already above the minimum flood control elevation, there was no time to quickly repair the main spillway. Furthermore, water could not be diverted through the Hyatt Powerplant or the river valve outlets either. The first was unusable because PG&E ceased supplying power to it, due to electricity towers and power lines being directly in the erosion paths of either spillway. The river outlets were also non-operational at the time according to the DWR (Messer, 2017):

> *"The River Valve Outlet System (RVOS) was available for use prior to February 7. It was flooded during the spillway incident with resulting damage to some of the operating and control components and had to be taken offline in February 2017. It was repaired in May 2017 and is currently available at a tested safe capacity of 4,000 cfs."*

-Cindy Messer, Letter to Coalition Members, June 7, 2017

At this point the Oroville Dam operators were facing a tough dilemma; either continue to release flows through the already damaged chute and cause further erosion, or risk using the untested auxiliary spillway. However, as the latter structure is ungated, if unchecked the dam itself would make that choice for them, as water would flow over the emergency spillway as soon as the surface elevation surpassed its crest, at 274.62 m (901 ft.). As such, a plan was formulated to continue letting small flows pass through the main spillway, while also preparing the area around the auxiliary spillway in case it would have to be put to use. To that end, workers began clearing the area downstream of this secondary structure, as well as placing large rocks at its foot to mitigate possible erosion.

Image 20. February 10th, 2017. Workers prepare the emergency spillway for use by placing large rocks at its foot. Source: Brian Baer, DWR.

At this point, the inflows into Lake Oroville increased tremendously, reaching the aforementioned peak of 5,392 m^3/s (190,435 cfs).

7.6 February 11-12: Water Flows Over the Emergency Spillway

On February 11, at 8:00 AM, surface elevation at Lake Oroville surpassed that of the emergency spillway crest, meaning that for the first time in the dam's history, water would pour over it.

According to data from CDEC, water poured over this ogee weir for just over 37 hours in total, as the surface level dropped below its crest elevation again on February 12 at 21:00 PM.

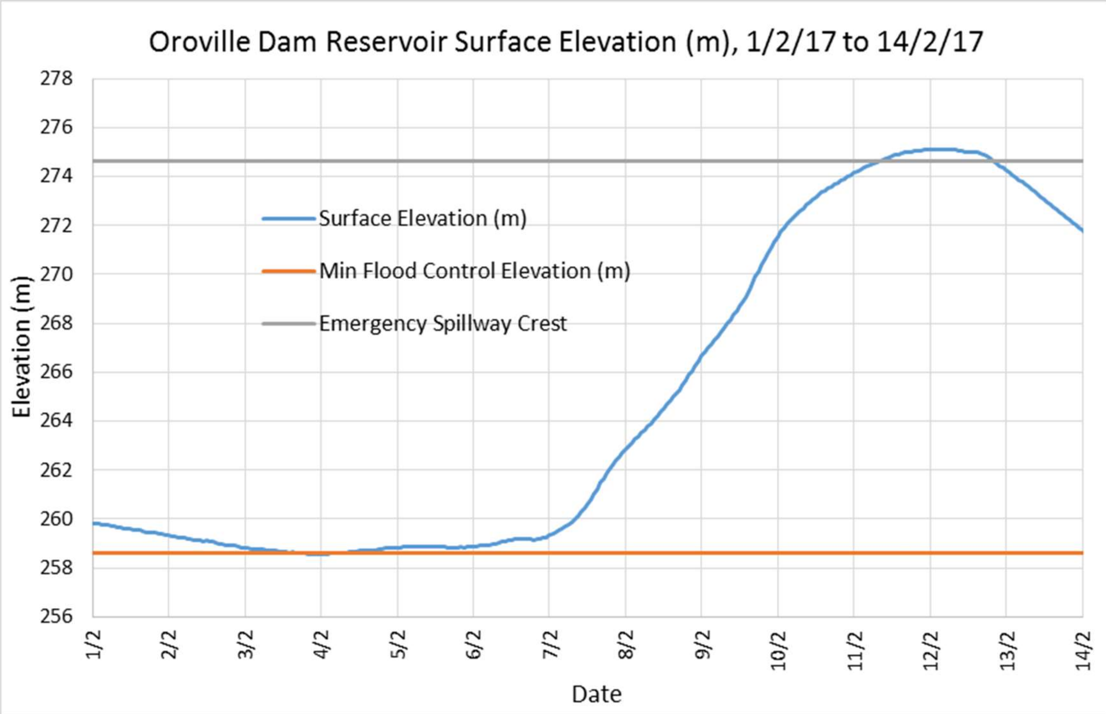


Figure 36. Oroville Dam reservoir surface elevation in meters, from February 1st, 2017 to February 14th, 2017.

An early image of flows over the emergency spillway crest can be seen below.

Image 21. February 11, 2017. Water flows over the Oroville Dam emergency spillway for the first time. Source: Zack Cunningham, DWR.

A rather interesting fact is that there is a parking lot just next to the emergency spillway, which is at a lower elevation, and thus is flooded by design whenever water pours over the weir. Furthermore, an access road located just below the structure was also subsequently flooded and quickly destroyed.

Image 22. February 11, 2017. Image of the flooded parking lot and access road located next to the emergency spillway. Source: Metabunk.org

Unfortunately, erosion downstream developed much more rapidly then anticipated. While the emergency spillway was only active for a very brief duration, and peak discharge did not exceed 400 m^3/s (15,000 cfs), large boils occurred downstream, destroying the access road below and threatening to damage the spillway crest itself by failure due to headcutting. One hole reached dangerously close to the structure, shown below.

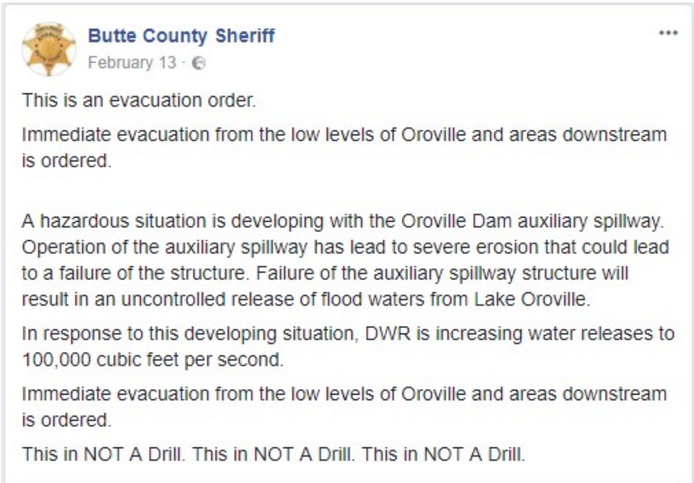
Image 23. February 13, 2017. The aftermath of the erosion caused by flows over the emergency spillway. Source: Randy Pench, Sacramento Bee.

This hole is more clearly visible in the zoomed in version below:

Image 24. February 13, 2017. Detail of a hole below the emergency spillway. Adapted from Randy Pench, Sacramento Bee.

It is hard to ascertain the exact distance between the edge of the closest hole and the concrete section of the emergency spillway, but a gross estimation can be made using the known length of the auxiliary structure, which is 527.3 m (1730 ft.) as a simple measuring scale. Overall, the distance between the edge of the hole and the spillway is approximately 20-25 m (65-82 ft.), which is dangerously close, and could have resulted in a failure of the emergency spillway if flows had not been immediately halted when the dam operators noticed the rapid erosion threatening to undercut the structure. If they had failed to do so in time, a void would have formed below the concrete weir, which would result in its failure, allowing 10 m (30 ft.) of water to flow freely through it and flood the downstream areas of Oroville and beyond.

The exact extent of the damage was not clearly visible when water was still pouring over the downstream hill on February 12, however, and thus local authorities, fearing the worst outcome, were forced to spring into action and order the evacuation of Oroville and other areas downstream of the dam, including Yuba City and Marysville.



Butte County Sheriff
February 13 ·

This is an evacuation order.

Immediate evacuation from the low levels of Oroville and areas downstream is ordered.

A hazardous situation is developing with the Oroville Dam auxiliary spillway. Operation of the auxiliary spillway has lead to severe erosion that could lead to a failure of the structure. Failure of the auxiliary spillway structure will result in an uncontrolled release of flood waters from Lake Oroville.

In response to this developing situation, DWR is increasing water releases to 100,000 cubic feet per second.

Immediate evacuation from the low levels of Oroville and areas downstream is ordered.

This in NOT A Drill. This in NOT A Drill. This in NOT A Drill.

Image 25. On the afternoon of February 12th, the Butte County Sheriff's office officially ordered the evacuation of Oroville and downstream areas through Facebook.

A map of the affected evacuation area can be found below.

Map 8. Map of the area ordered to be evacuated after the Oroville Dam spillway incident. Adapted from jpedderDRP, National Geographic, Esri, DeLorme, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, and increment P Corp., using ArcGIS software.

More than 180,000 people in total live in this area, and thus this evacuation order made the Oroville Dam spillway incident headline news worldwide (BBC, 2017). Most of the evacuees were sent to Chico, the nearest northern city that would be unaffected from any possible flooding.

The California Department of Water Resources responded to the evacuation order by immediately increasing outflow releases from the main spillway to 2,830 m^3/s (100,000 cfs). This would drastically lower the surface elevation and stop flows over the emergency spillway and any resulting erosions there, at the cost of causing irreparable damages to the main spillway. Luckily, despite the conditions, the upper portion of the main spillway was able to release these discharges without causing further upstream erosion. However, the hill downstream of the initial hole would be quickly eroded away from high velocity flows. Images of the unfolding damage can be seen below.

Image 26. February 11th, 2017. Water flowing over the damaged main spillway. Some of the discharge is flowing through the initial hole under the chute's left wall, creating a new channel. Source: Florence Low, DWR.

Image 27. February 13th, 2017. 2,830 m^3/s (100,000 cfs) flowing over the main spillway. The chute's upper portion remains undamaged, but erosion quickly develops downstream. Source: Kelly M. Grow, DWR.

Image 28. February 15th, 2017. An aerial view of Oroville Dam's damaged spillways. Source: Dale Kolke, DWR.

Since the main spillway was able to withstand these high discharges without eroding upstream, a decision was made to continue these outflows for a long time, up to approximately the afternoon of February 16th, then steadily decrease them, finally reducing them to zero once the surface elevation was low enough to be considered safe, at which point efforts could be made to assess the damage and work on clearing resulting debris. A detailed hourly inflow/outflow hyrdograph of these critical moments, as well as an hourly graph of Lake Oroville's surface elevation can be found below.

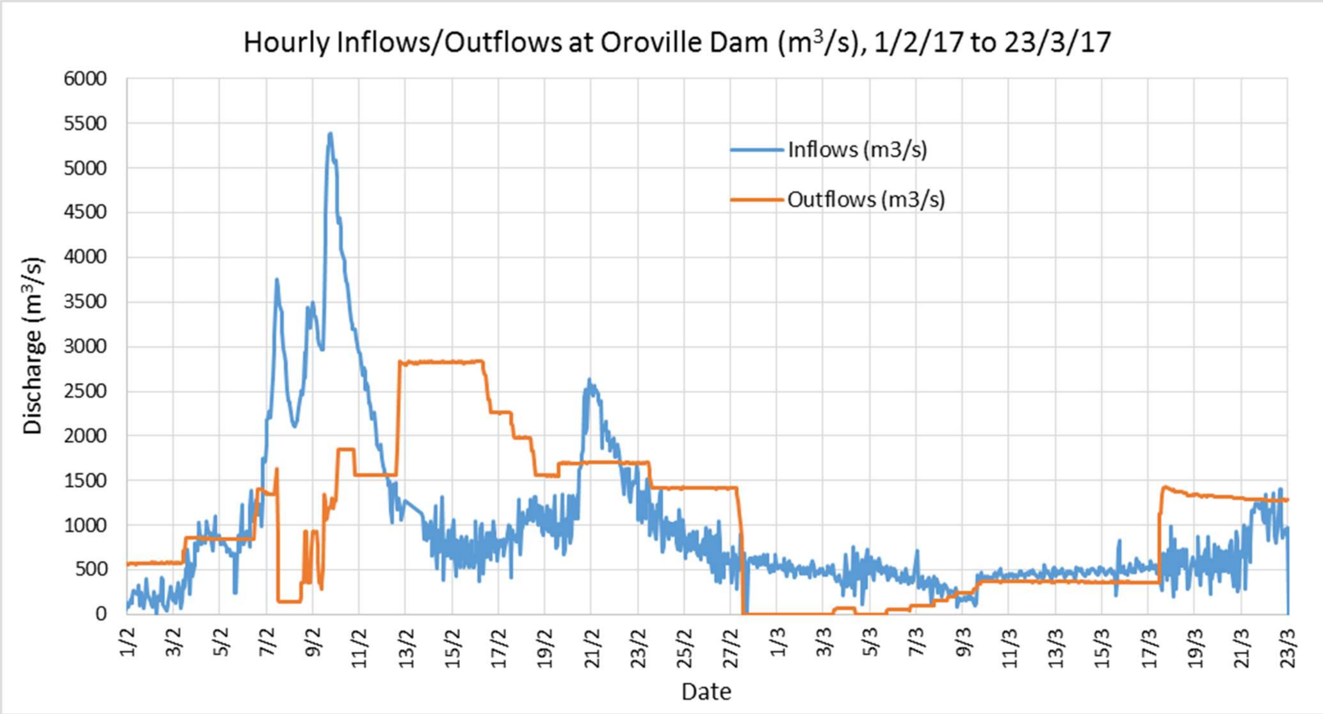


Figure 37. Hourly Inflows and Outflows at Oroville Dam in m³/s between 1/2/17 and 23/3/17.

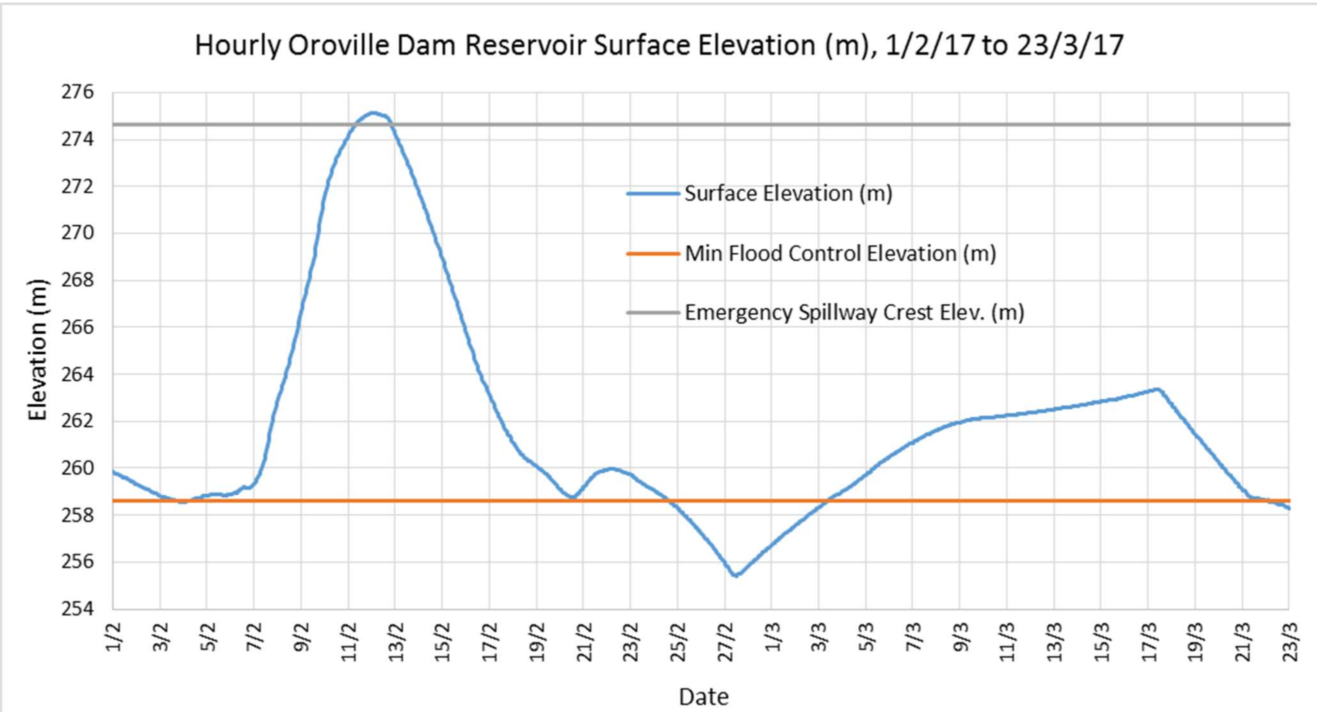


Figure 38. Hourly changes in Lake Oroville's surface elevation in m, compared to the minimum flood control elevation and the emergency spillway crest elevation, from 1/2/17 to 23/3/17.

7.7 Repair Efforts

On February 13th, once flows over the emergency spillway stopped, an attempt to quick remedy the downstream erosion began, in case it would have to be put to use again soon. Since the emergency spillway access road was destroyed, helicopters were used to transport large bags of rocks up to the parking lot next to the emergency spillway, and workers at the dam placed them below the auxiliary structure, covering any holes that had formed from the previous day's erosion, then pouring concrete on top to create a more solid base for potential future flows, which fortunately did not occur.

Image 29. A helicopter transports a bag of large rocks over to the emergency spillway. Source: Florence Low, DWR.

On February 27th, outflows from the main spillway briefly stopped. An image below shows the aftermath of the incident.

Image 30. February 27th, 2017. Aerial view of the Oroville Dam main spillway. Large sections have eroded away and ended up in the Feather River as debris. Source: Dale Kolke, DWR.

A large scale effort began with the purpose of quickly clearing debris from around the spillway and working on repairing it. The holes around the emergency spillway were also almost completely filled up with rocks and concrete until that point.

After a contractor was selected for the repair project, work on the main spillway chute began. Repair work consisted of three phases: First, shortcrete was applied under the upper portion of the main spillway to halt further erosion and allow small flows to pass over it.

Image 31. March 5th, 2017. Worker applies shortcrete under the main spillway chute's upper portion. Source: Kelly M. Grow, DWR.

In the second phase, blasting was used to quickly remove the entire lower segment of the main spillway chute, as it is intended to be replaced by a new structrure.

Image 32. May 30th, 2017. Blasting is used at the lower portion of the main spillway chute. Source: Kelly M. Grow, DWR.

In the third and final phase, the upper segment of the main spillway is repaired, whereas the lower portion is replaced by a new one, built partly out of structural concrete and roller-compacted concrete (RCC). Estimated cost for the project is at around $500 million, and is estimated to conclude by November. A recent image of the near completed spillway recently surfaced, shown below.

Image 33. October 13th, 2017. View of the Oroville Dam main spillway chute repair work. Source: Kelly M. Grow, DWR.

For the emergency spillway, the current plan is to build a concrete splashpad and an underground secant pile cut-off wall immediately downstream, which should prevent any headcutting from occurring if the weir was ever used again. A detailed graphic of current construction plans is available below.

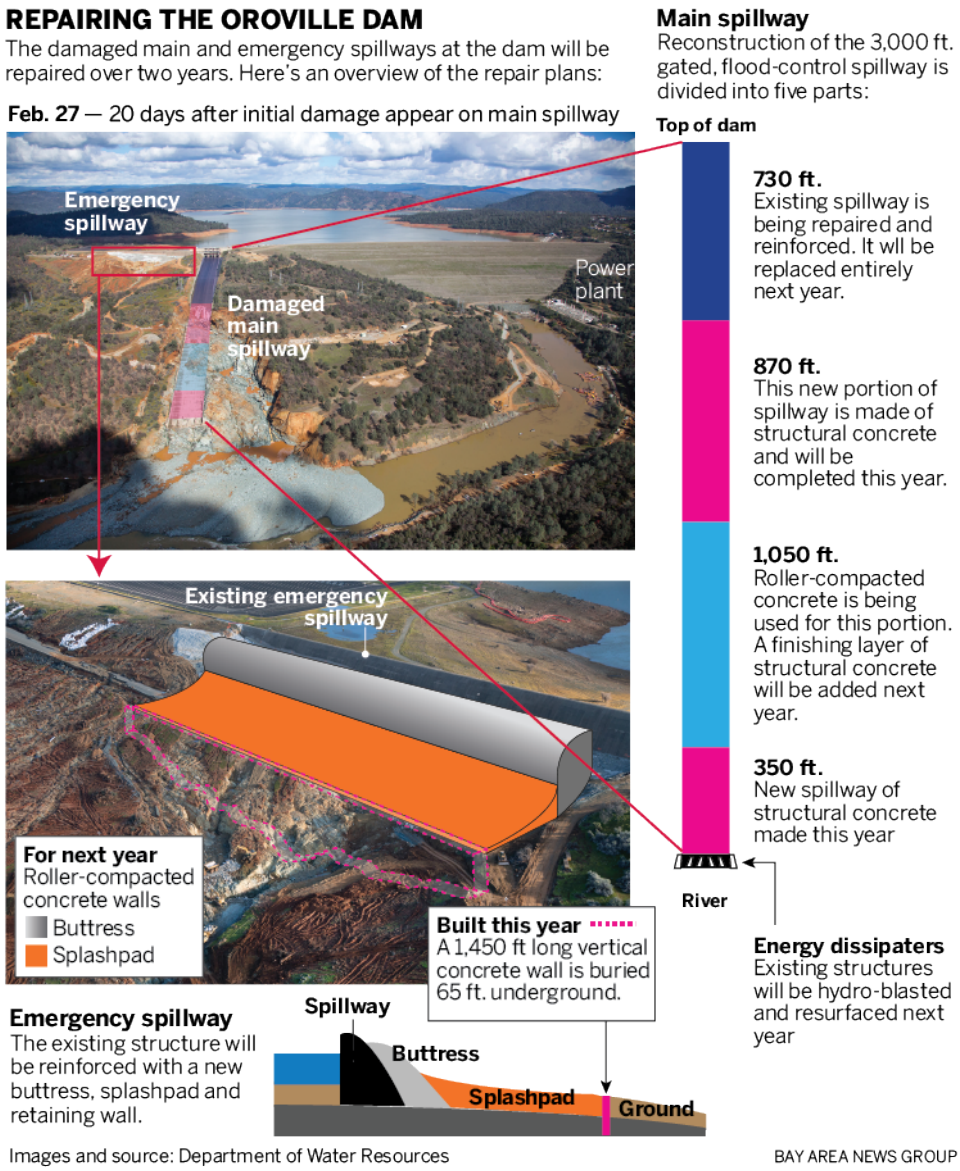


Image 34. Current repair plans for the Oroville Dam spillways. Sources: California Department of Water Resources, Bay Area News Group.

This repair effort was not made without criticism, however. Various dam experts, including Scott Cahill (Cahill, 2017), argued that blasting at the Oroville Dam site is extremely dangerous given the poor geological conditions and the inability to deal with any subsequent erosions or further damages occurring from this procedure. Furthermore, the now vindicated Friends of the River group, which had requested for a full armored concrete emergency spillway back in 2006, is still asking for one, and considers that the currently planned structure will be inadequate (Schnoover, 2017).

7.8 Consequences of the Spillway Incident

The Oroville Dam spillway incident became known worldwide mostly for the impact it had on the downstream communities; out of nowhere, suddenly 180,000 people were ordered to leave their homes after assurances that the February floods were routine and would be handled without any issues. In the engineering and scientific communities, it is an excellent case study, as it is a dam failure that occurred within normal operating conditions. The dilemma posed to the dam operators on February 10, about choosing to use the emergency spillway or risk further damaging the main chute, is of particular importance. Not many dams have the ability to divert flows to a secondary structure if the main spillway fails. Yet in this particular case, an emergency spillway meant as a a sacrificial plug in order to avoid overtopping of the main dam embankment ended up being a weakness, not a feature. With the current conditions, if the reservoir surface elevation is to exceed that of the emergency spillway crest, water will always flow over it first, instead of over the dam, and the ridge between the spillways and the main embankment protects the latter from any immediate damage.

Image 35. A sketch of the ridge that protects the Oroville Dam main embankment from erosion. Adapted from Google Earth (2017).

In any case, the consequences of the Oroville Dam spillway incident will have a great effect on the local communities in the years to come. Fortunately, there were no known fatalities as a cause of the event or the subsequent evacuation process. However, there are a multitude of other negative impacts. First, economical impacts due to the structural damages caused, which require immediate and expensive repairs. Indirectly, the local population suffered an economical blow due to the evacuation, as they lost any wages they could have earned during those days, and the local real-estate market should suffer from the negative press which throughout the year has highlighted the dam's lack of safety and the potential flooding risk of any areas downstream of the dam. Furthermore, from an ecological standpoint, the local Feather River fishery's ecosystem suffered from the influx of muddy water as a result of flows through the damaged main spillway.

At this point, an effort of the local community should be highlighted. On February 10th, before the incident was in full effect, the California Department of Fish and Wildlife moved 4 million baby salmon from the Feather River Hatchery located near Oroville Dam to the downstream Thermalito Afterbay Complex, shown below.

Image 36. February 10th, 2017. Four million baby salmon are transferred from the Feather River Hatchery to the Thermalito Afterbay Complex. Source: Kelly M. Grow, DWR.

This rapid mobilization, together with help from other agencies, helped save most of the hatchery's total fish population of 8 million young salmon.

Also, the recreational capabilities of Oroville Dam have been harmed. As long as repairs are underway (which may continue well into 2018), the reservoir elevation will be kept at extremely low levels, much further below the minimum flood elevation, which will negatively affect structures such as the spillway boat ramp and the downstream Thermalito Diversion Pool, recently built boating and paddling facilities that will now be closed for the next several years (Stork, et al., 2017).

Finally, the emotional impacts on the downstream communities must also be mentioned. Local residents criticized local authorities for not warning them of danger earlier, and after this incident many of them feel justifiably unsafe and fear a potential similar event occurring in the future, with more devastating consequences to their lives and properties (BBC, 2017).

8. Possible Causes of the Spillway Incident

8.1 A Scientific Approach

When examining the causes of a real event such as the Oroville Dam spillway incident, lacking the ability to perform an on-site forensic investigation, it is tempting to simply look at photographs of damaged structures, and attempt to gather clues directly from them. However, this is less of a scientific approach and more that of a typical conspiracy theorist. To avoid jumping to unfounded conclusions, it is of the utmost importance to follow the steps of a proper scientific method: formulating a question, doing background research, testing with experiments (or models) and troubleshooting their results.

Based on the evidence already gathered, it is possible to make several hypotheses for the possible causes of failure for both spillways.

8.2 Emergency Spillway

It is much easier to determine the cause of the near failure of the emergency spillway due to the fact that it was actuated for a very brief duration under constant supervision, as authorities were already alerted of the situation. While water was pouring over the concrete weir without a problem, it was the surrounding conditions that posed a threat.

Already from the documents describing Oroville Dam's construction, the following facts are known:

1) The emergency spillway was untested, even in the model studies conducted by the US Bureau of Reclamation. (US Bureau of Reclamation, 1965)

2) The area downstream of the emergency spillway was not cleared. (California Deparment of Water Resources, 1974)
3) Regarding the emergency spillway foundation excavation, it continued 3 m (10 feet) deeper than expected, in order to reach foundation rock that met the design criteria. (California Deparment of Water Resources, 1974)

While it is known that this concrete overpour weir was built on a solid foundation, no effort was made to secure that the downstream ridge would be able to accommodate flows passing over it without significant erosion occurring as a result. This could have been acceptable if this structure was truly used as an emergency measure (i.e. any outflows from it not being factored into hydrologic design calculations, using only the main structure's design capacity instead), but this is not the case. According to original design specifications (California Deparment of Water Resources, 1974); (US Army Corps of Engineers, 1970); (US Bureau of Reclamation, 1965), the main spillway alone is built to withstand the Standard Project Flood inflow peak of 12,700 m^3/s (450,000 cfs). This flood has yet to occur, but is significantly below the Probable Maximum Flood (PMF).

According to (Morrison-Knudsen Engineers, Inc., 1986), a high risk structure such as Oroville Dam should be able to withstand the PMF. All PMF analyses so far (US Army Corps of Engineers, 1970); (DWR, 2004) have included the emergency spillway in their calculations, and in fact, in the event of the PMF, the emergency spillway is expected to reach outflow discharges of around 10,000 m^3/s (350,000 cfs). Seeing as erosion threatened to cause structural failure at less than 420 m^3/s (15,000 cfs), the spillway's ability to withstand PMF-level discharges is questionable. In any case, this warrants the need for the structure to be properly armored with concrete and considered to be an "auxiliary" spillway, not an "emergency" one. This has been repeatedly requested by the community (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006); (Stork, et al., 2017); (Schnoover, 2017), and has yet to be fully implemented.

8.3 Reservoir Surface Levels Prior to the Flood

When posing the question of why Oroville Dam was capable of withstanding the previous floods of 1986 and 1997, and not the 2017 event, one is prompted to also examine the surface elevation levels prior to each flood.

An attempt is made to compare Oroville Dam reservoir surface levels shortly before and after each of the three recent flood events, occurring in 1986, 1997, and 2017.

In the graph below, the y axis represents surface elevation in meters, whereas the x axis represents up to 240 hours (10 days) before and after peak inflow. Hour 0 is the hour during which peak inflow occurred for each event.

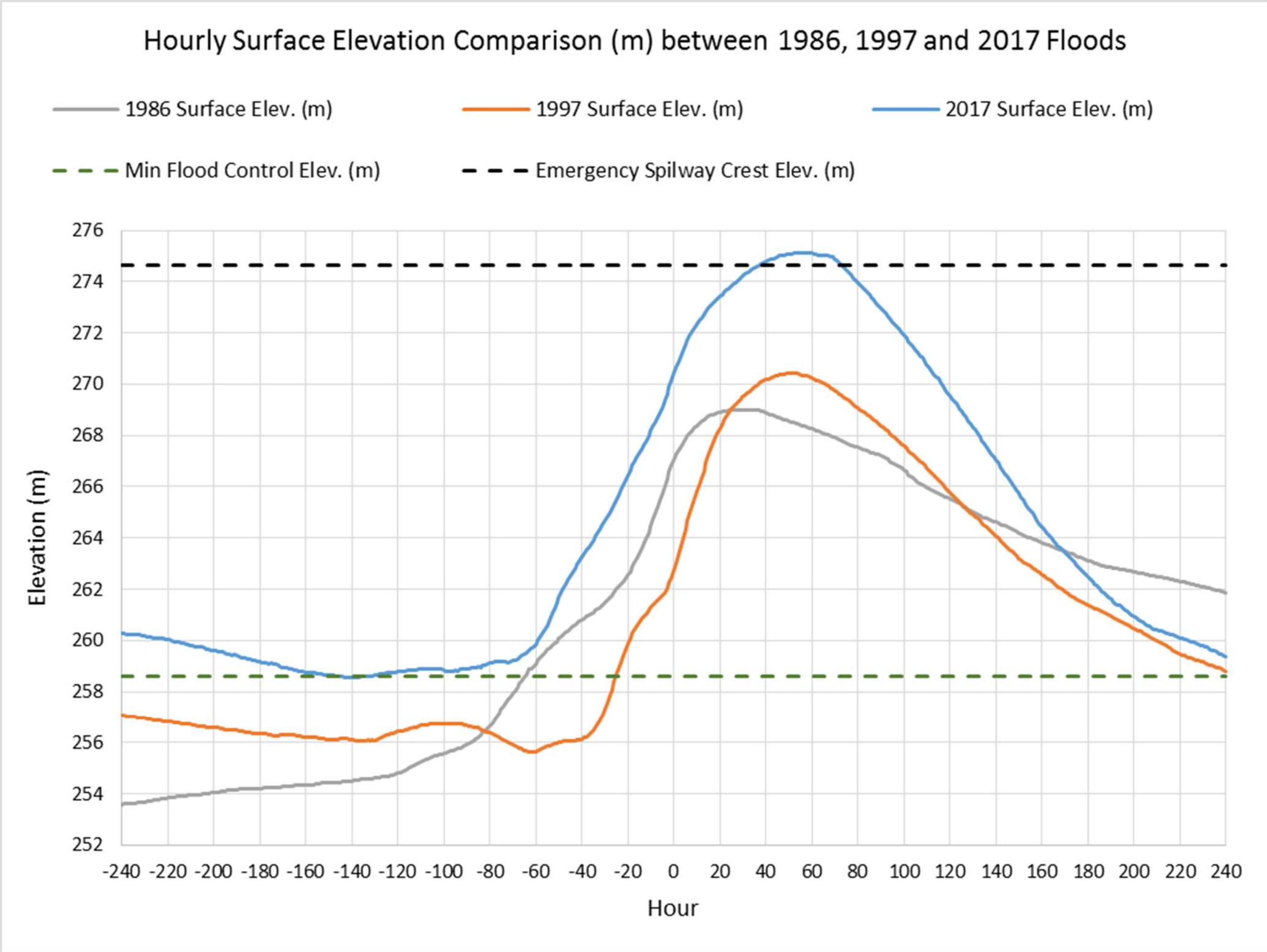


Figure 39. Hourly comparison of Oroville Dam reservoir surface elevations 10 days before and after the peak inflow of the 1986, 1997, and 2017 flood events.

Of course, no two flood events are the same and they all impact Oroville Dam in subtly different ways, but this comparison contains clues on what went wrong during the 2017 spillway incident.

Notably, while the 2017 peak inflow is the lowest of the three major flood events, its surface elevations are the highest. This is due to two factors. First, as is clear from the graph, shortly prior to peak inflow, surface elevation during 2017 was higher than that of previous floods. Already, this has a negative impact on flood management. While this elevation is below the minimum limit specified by the flood control manual (US Army Corps of Engineers, 1970), the 2017 flood is actually harder to manage than that previous events. This is partly why despite it not being a record flood, this event came close to causing severe damages to Oroville Dam's key structures once the main spillway failed.

It would be easy at this point to say in hindsight that the surface elevation should never have been allowed to be this high within a wet period, and that outflows from the main spillway should have been raised during the previous January 2017 flood instead of being next to zero. However, this might have caused the main spillway to fail sooner, and in the end result in more severe damages after the subsequent February event.

8.4 Main Spillway

Attempting to detect what caused the initial failure of the main spillway is a much more complicated task, as due to the nature of the incident, very few pictures are available showing the initial chute hole that was spotted on February 7^{th}. Any physical evidence that could have been gathered from the scene at the time has been likely washed away from the subsequent discharges that eroded away the bottom half of the chute and much of the downstream ridge. As was mentioned earlier, simply looking at pictures of the February 7^{th} chute damage is not enough, and can lead to forming unbased conclusions. Thus, prior to studying these pictures, further background research is required.

A dam inspection guide (Morrison-Knudsen Engineers, Inc., 1986), lists potential incidents that can occur to spillway concrete chutes, and possible causes based on studies of previous similar events. More specifically, the following defects mentioned in the guide are directly related to the Oroville Dam main spillway chute.

1) Cracking of concrete in floor slabs. Visible on casual inspection when concrete is dry, possibly caused by temperature changes or inadequate reinforcement.
2) Damaged concrete. Possibly caused by cavitation or erosion due to irregularities or rough surface.
3) Lifted slab panels. Indicated by vertical offsets in joints, possibly caused by poor drainage under slabs, and/or inadequate anchoring of slab to foundation.

Futhermore, the following additional factors are considered:

4) Geological conditions below the spillway chute.
5) Possible damage due encroaching vegetation around the main spillway chute.

All of these factors are examined below.

8.4.1 Cracking of Concrete in Floor Slabs

Based on previous inspection reports and other sources (California Deparment of Water Resources, 1974); (Federal Energy Regulatory Commission, 2009); (Federal Energy Regulatory Commission, 2014), it is known that cracks had previously occurred in the main spillway chute's floor slabs. It is possible to compare pictures of the 2013 repairs to the 2017 damage.

Image 37. Comparison of 2013 chute repairs and February 6th, 2017 initial chute damage. Sources: Unknown and Kelly M.Grow, DWR.

Based on the locations of a tree and a drain on the main spillway chute wall which are common on both photographs, it is clear that the 2013 repairs took place just above the location of the 2017 hole. It is possible that concrete cracking occurred in both events and led to the pictured damage.

Unfortunately, the cause of the 2013 cracking is unknown. The crack widths are not specified, but they could be a result of either temperature changes or inadequate reinforcement. The first cause is unlikely to have caused severe damages on its own, as small cracks have been filled whenever they occurred, and the flood control manual contains an aforementioned rule regarding how quickly flows are to be increased and decreased. Specifically, they are not to be increased more than 280 m^3/s (10,000 cfs) or decreased more than 140 m^3/s (5,000 cfs) in any given 2-hour period (US Army Corps of Engineers, 1970). This rule was likely placed in order to better regulate downstream flows, as well as limit temperature changes within the spillway chute concrete. However, analysis of the outflow data given by the CDEC (California Department of Water Resources, 2017) shows that this rule was maintained prior to the 2017 incident. The other cause is inadequate reinforcement, which can only be specified with an on-site inspection.

8.4.2 Possible Cavitation – 1-D Water Surface Profile Analysis

One of the possible causes of the initial damage to the concrete chute floor is cavitation. In order to better understand this cause, extensive examination of the USBR hydraulic model study of the main spillway (US Bureau of Reclamation, 1965) is required. Furthermore, comparing this data to a simple mathematical model of the main spillway chute could help find possible clues.

An attempt was made to create a model of the main spillway chute using HEC-RAS, but was impossible due to the steep curves of the chute's lower section, which result in analysis errors due to software limitations. Instead, a simple mathematical model is constructed in Microsoft Excel which uses the same iterative procedure to simulate 1-D steady flow, known as the standard step method (US Army Corps of Engineers, 2016).

In order to construct this model, some additional assumptions must be made, which are analyzed below.

Based on the USBR main spillway chute profile, its main rectangular concrete section is 178.67 feet (54.46 m) wide, begins at Station +13 00 (1,300 feet past the beginning of the approach channel) and ends at Station +43 00, just before the terminal structure with the concrete chute blocks. As such, this main section is exactly 3,000 feet (914.4 m) in length, and only this part of the main spillway is modeled. To avoid confusions between the USBR calculations and those of the model, the entire model is constructed using American unit measurements (distance in feet, discharge in cfs, etc.).

To calculate flows, Manning's *n* coefficient is additionally required. Unfortunately, there is no mention of the specific coefficient used for the hydraulic calculations of the final chute in (US Bureau of Reclamation, 1965). However, a profile drawing of an earlier model describes a lined concrete channel with an *n* value of 0.013. Based on this and the HEC-RAS manual specifications, an *n* value of 0.014 was selected for the model.

Furthermore, in the interest of time and with the intent of keeping the mathematical model as simple as possible, critical flow depth was assumed at the chute's beginning for every discharge profile, instead of the true depth which is partially controlled by the flood control outlet gates. However, as is evident later, this did not have a significant impact on the results.

Four discharge profiles were created, in accordance with those of the USBR model study: 20,000 cfs (566 m^3/s); 50,000 cfs (1,416 m^3/s); 100,000 cfs (2,832 m^3/s); and finally 277,000 cfs (7,484 m^3/s), which is the main spillway's design capacity. Water surface profile views of the chute for each discharge profile are plotted below, with additional data label at the exact point where the 2017 hole occurred (Station +33 00).

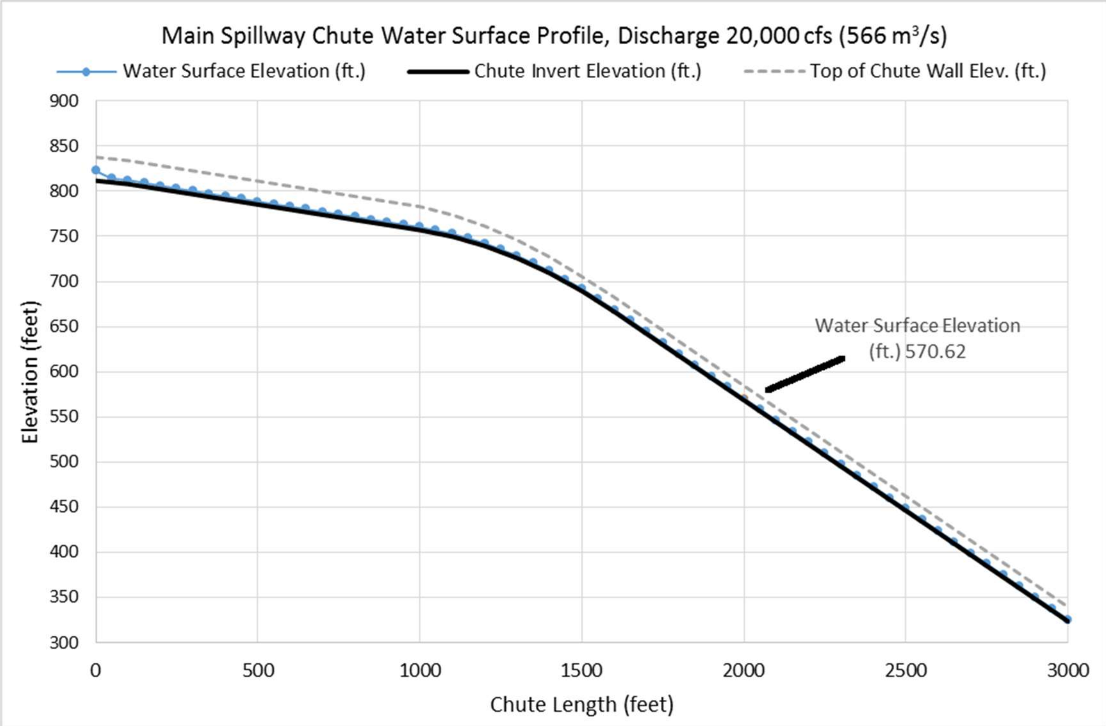


Figure 40. Oroville Dam main spillway chute water surface profile, discharge 566 m³/s (20,000 cfs)

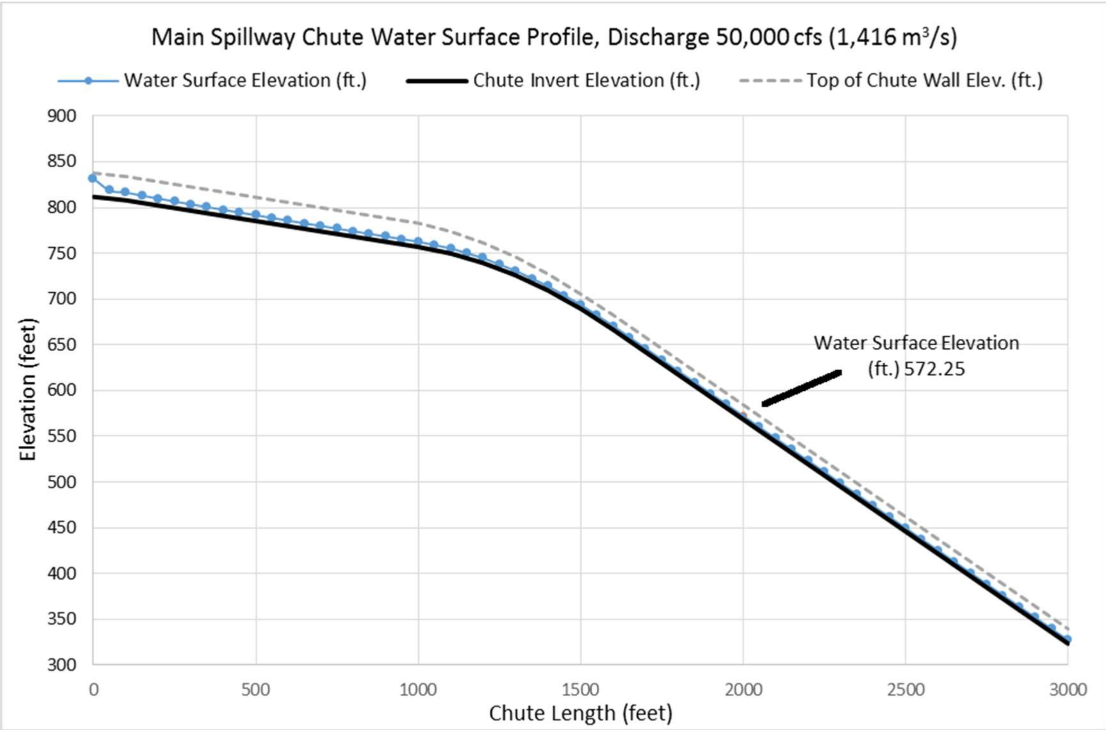


Figure 41. Oroville Dam main spillway chute water surface profile, discharge 1,416 m³/s (50,000 cfs)

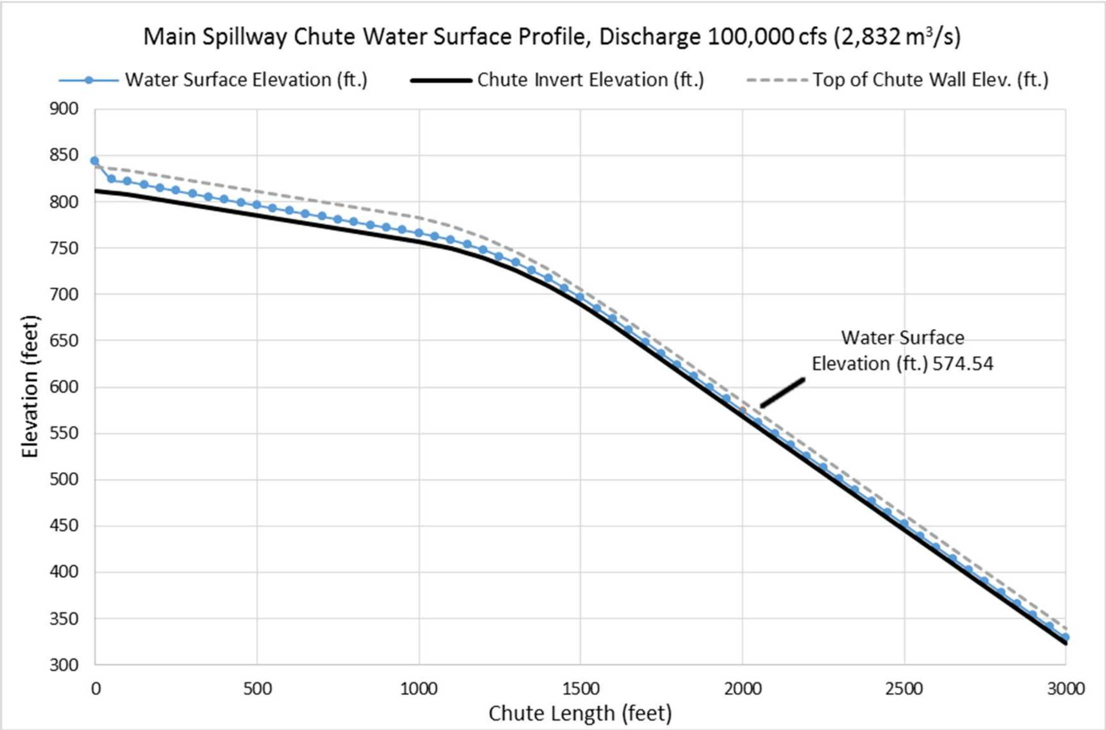


Figure 42. Oroville Dam main spillway chute water surface profile, discharge 2,832 m^3/s (100,000 cfs)

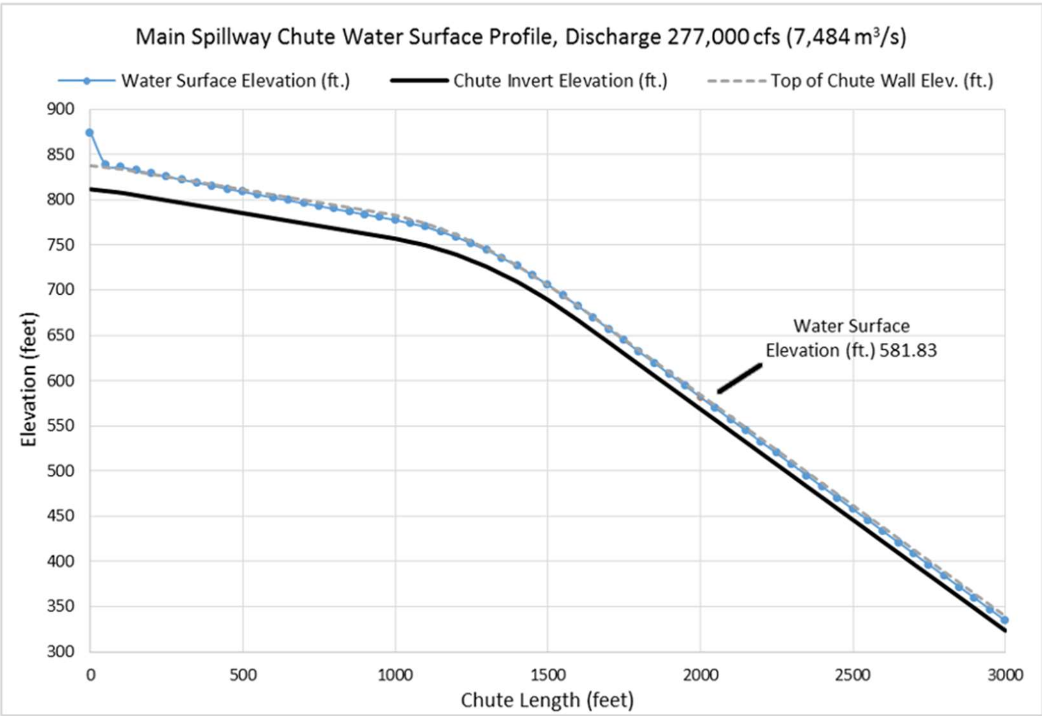


Figure 43. Oroville Dam main spillway chute water surface profile, discharge 7,484 m^3/s (277,000 cfs)

From the chute flow analysis, it is clear that the initial assumption of critical flow depth at the chute's beginning does not negatively impact the results significantly, as due to the chute's design, flow depth quickly approaches normal depth with a standard S2 curve for supercritical flow (US Army Corps of Engineers, 2016). For low discharge profiles, normal depth is reached fairly quickly, and only when the spillway is running at maximum capacity, 7,484 m^3/s (277,000 cfs) does the flow reach normal depth close to the chute's end. No surface flow irregularities are immediately apparent from this analysis, indicating that cavitation is probably not the initial cause of the of main spillway's failure.

This simple model can also give an estimate of flow velocities. When discharge is at 2,832 m^3/s (100,000 cfs), as it was on February 12th, 2017, average flow velocity at the point where the main spillway failed (Station +33 00, chute length 2,000 feet) reaches an estimated 29 m/s (96 fps), or 105 km/h. This explains the intense force of the water flow; once the initial hole was opened in the spillway, the stream was easily able to erode away large sections of the chute and the downstream ridge.

However, before this emergency outflow was necessary, on February 6th shortly before the spillway failed, it was operating with a discharge of approximately 1,416 m^3/s (50,000 cfs). Under these conditions, estimated velocity at the failure point is 23 m/s (77 fps), or 84 km/h. Indeed, due to how flow dynamics work, cutting discharge down to half does not reduce flow velocity to half as well, and these speeds are surely capable of severly damaging the chute once an irregularity emerges among the concrete floor slabs.

Finally, from this analysis, a problem emerges when discharge reaches the maximum of 7,484 m^3/s (277,000 cfs), as the flow overtops the concrete chute walls, once near the beginning of the chute, and secondly at the curved section 1500 feet (500 meters) into its length. The first overtopping is likely a result of the initial critical depth assumption, and can be disregarded. However, the second overtopping warrants further research. When comparing this water surface profile to that of the USBR model study for a 8,269 m^3/s (292,000 cfs) discharge, the flow depth of the simple mathematical model is slightly higher instead of lower. A number of factors could have caused this, including the initial assumptions of critical flow depth and Manning's *n* coefficient value. However, even so, this highlights a possible risk of overtopping occurring in the spillway chute if such outflows were ever necessary, and pictures of the 1997 event (Image 10); (Image 11) show how close the chute came to overtopping while operating at a much lower discharge. It must be stated at this point that the Department of Water Resources is considering raising the chute walls of the new main spillway (California Department of Water Resources, 2017) for this very reason.

Detailed tabular output of the above water surface profile analysis can be found in Appendix C.

8.4.3 Lifted Slab Panels – Drain System Deficiencies

Unfortunately, due to the nature of the incident and the measures that had to be taken to ensure Oroville Dam’s safety, if the initial cause of the main spillway chute’s failure was slab uplift due to a fault in the drain system, the only available evidence can be found in pictures taken shortly before and after the February 6 chute hole was spotted, as any physical evidence was subsequently eroded away by the February 12th outflows. However, by conducting background research, the following factors are discovered about the main spillway’s drain system and the concrete chute slabs (California Deparment of Water Resources, 1974); (Federal Energy Regulatory Commission, 2009); (Federal Energy Regulatory Commission, 2014); (Division of Safety of Dams, CA DWR, 2015):

1) The invert slabs have a minimum thickness of 380 mm (15 inches), are anchored to rock with grouted anchor bars, and are provided with a system of underdrains. (California Deparment of Water Resources, 1974)
2) The initial drain system plan ended up being significantly altered during construction. After a recommendation by the Oroville Dam Consulting Board, the original horizontal pipe drains under the chute were enlarged and placed in a herring-bone pattern. The collector system operating in line with the chute was also enlarged and modified so as to enhance its capacity and self-cleaning ability. (California Deparment of Water Resources, 1974)
3) The last official inspection of the main spillway chute’s full length took place in 2015 (Division of Safety of Dams, CA DWR, 2015). At the time, no structural deficiencies were detected. An additional inspection took place in 2016 (Division of Safety of Dams, CA DWR, 2016), but the spillway chute was only examined from the top of the FCO outlet structure, not up close like in 2015. A reason for this is not specified.

Furthermore, a comparison of pictures of the spillway shortly before the February 6 hole was discovered yield additional clues.

Below is a comparison of two pictures of the main spillway chute, taken shortly before the February incident. The first was taken on January 11th, 2017 and the second on January 27th of the same year.

Image 38. Views of the Oroville Dam main spillway chute. Left photo taken January 11th, 2017, and right photo taken January 27th, 2017. A red arrow points to the location of the initial chute failure. Sources: Kelly M. Grow, DWR and Bill Husa, Chico Enterprise-Record.

While these pictures were only taken within 16 days of each other, there are significant differences in the spillway chute. A center section of the chute's concrete floor appears dry on the right-hand picture, despite flows passing over the rest of the structure. This indicates possible irregularities among the floor slabs. Furthermore, the fact that this dry patch is not visible in the photo taken earlier, could possibly mean that a possible slab uplift occurred near the red arrow's location, diverting small water flows around it instead of over it.

Outflow conditions must also be taken into account. As already described in a previous analysis of the January 2017 rain storm (Figure 26; Figure 28), outflows around January 11th were low, whereas around January 27th, discharges were approximately 283 m^3/s (10,000 cfs). If floor slab uplift caused the spillway chute damage, it must have occurred around this time.

In addition, below is a side by side comparison of the same pictures, but zoomed in to better show the drains on the chute's left wall.

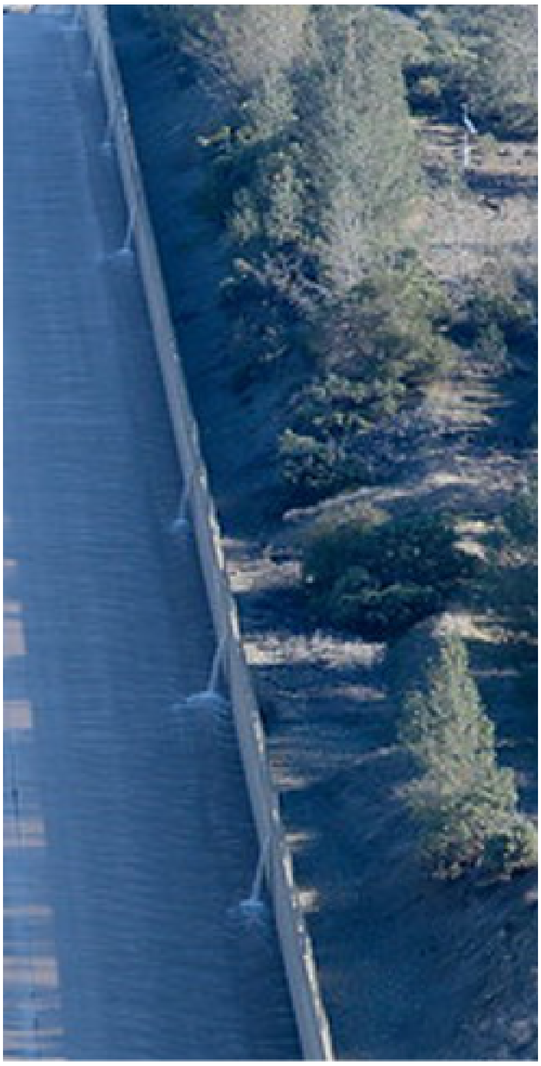



Image 39. Side by side comparison of drains in the spillway chute's left wall, left photo taken January 11, 2017 and right photo taken January 27, 2017. Sources: Kelly M. Grow, DWR and Bill Husa, Chico Enterprise-Record.

This comparison reveals two clues: Firstly, water is coming out of the drains under pressure, which is not according to design specifications, and secondly, discharge from these drains significantly increased in a short time, once flows from the January flood filled up the Oroville Dam reservoir. This is a telltale sign of a buildup of excess water occurring beneath the spillway, which could apply significant forces to the concrete slabs from below and cause them to uplift (Morrison-Knudsen Engineers, Inc., 1986). Additionally, the January 27 photograph shows the drains on the opposite wall operating under pressure as well.

8.4.4 Geological Conditions Beneath the Main Spillway Chute

Based on aforementioned data acquired through background research, the following information is available about the geological conditions beneath the main spillway chute:

1) Geology immediately in and around Lake Oroville is comprised mostly of what is called the “Bedrock Series”. This consists mostly of metavolcanic and pyroclastic rock, such as amphibolite. Above this bedrock lie various younger sedimentary rocks such as shales, dolomites, etc (Koczot, Jeton, McGurk, & Dettinger, 2005); (Jennings, Strand, & Rogers, 1977); (Freeze & Cherry, 1979).
2) Blasting was used for almost 90% of the chute foundation, in order to reach grade. The remaining amount consisted of the removal of several seams of clay located in the foundation, and a few areas where the slope failed (California Deparment of Water Resources, 1974).
3) The slopes in the flood control outlet section were of a lower quality rock than initially presumed, and several large seams ran parallel with the main spillway chute. The countermeasure that was applied was the replacement of planned anchor bars with grouted rock blots, pigtail anchors, and chain-link covering of the area’s surface (California Deparment of Water Resources, 1974).

The fact that the main spillway chute was built atop rock that required blasting to excavate would mean that it is suitably hard to serve as foundation for the concrete chute sections. However, pictures of the initial spillway failure reveal more information about this foundation rock.

Image 40. February 7th, 2017. Side view of the initial spillway chute failure. Source: Kelly M. Grow, DWR.

Based on this photograph, it appears that the foundation rock is indeed comprised of the metavolcanic materials mentioned previously. However, this particular section of bedrock appears highly fractured and heterogeneous. There is a significant variance of color in the formations, indicating different degrees of weathering. Furthermore, due to the orientation of the seams, the rock is expected to erode away in large chunks, not in sheets. It is also possible that water was able to seep through cracks in the weaker, more weathered sections of rock and undermine the chute from below.

8.4.5 Vegetation Around the Main Spillway Chute

Next, a possible cause for the initial main spillway chute failure could be a possible undermining of the ground around the structure caused by encroaching vegetation. If any trees of large bushes are allowed to grow next to a spillway, their roots could negatively impact the ground below it.

Previous inspection reports by the Division of Dam Safety give a rough estimate of the assumed standards for vegetation removal. A 2013 report (Division of Safety of Dams, CA DWR, 2013) mentions a need for further vegetation removal, which was accomplished prior the next report in 2015 (Division of Safety of Dams, CA DWR, 2015). This report also cites the need to remove an aforementioned “lone tree” (Image 15) This lone tree can serve as a sort of vegetation removal borderline. If vegetation around the spillway during the later years is behind this theoretical line, it can be assumed that an inspection would consider the conditions to be suitable. Indeed, the most recent 2016 inspection report assumes just that (Division of Safety of Dams, CA DWR, 2016).

Looking back at early January pictures of the spillway chute (Image 38), the vegetation is close to the aforementioned theoretical border, but does not exceed it. One could argue that this level of vegetation is still not up to standards, however.

In any case, it is not possible to confirm if this is a possible cause of the initial failure without conducting an on-site forensic investigation to detect possible roots underneath or near the concrete chute. Unfortunately, if such evidence existed, it has likely been removed by the erosive flows of February 12th and beyond.

9. Possible Solutions and Alternative Design Methods

9.1 Weaknesses of the Probable Maximum Precipitation and Probable Maximum Flood Methods

Until now, known flood control studies for Oroville Dam and the Feather Basin have attempted to determine the Probable Maximum Flood (PMF) for Lake Oroville, based on the Probable Maximum Precipitation (PMP). According to (Morrison-Knudsen Engineers, Inc., 1986); (DWR, 2004), all dams which are considered high-risk structures like Oroville Dam must be designed to withstand the PMF. Analysis is also conducted to determine the Standard Project Flood (SPF), a flood event weaker than the PMF, yet more akin to what engineers would call a “design flood”. As stated in the Oroville Dam flood control manual (US Army Corps of Engineers, 1970), the main spillway chute was designed with this flood in mind, in order to limit downstream flows to 5,094 m^3/s (180,000 cfs). According to all recent flood control studies, Oroville Dam is also capable of withstanding the PMF, although in most scenarios, the larger part of the outflow is expected to be routed through the emergency spillway (US Bureau of Reclamation, 1965); (US Army Corps of Engineers, 1970); (California Department of Water Resources, 2017).

However, the PMP-PMF analysis has several flaws. From a theoretical standpoint, the PMP suggests that there exists a theoretical upper limit of precipitation, which is simply not true. Nature is not bounded by numerical constraints, and the study of a brief history of available data cannot generate a true possible maximum value of precipitation. According to (Benson, 1973), the only merit of the PMP value is that it a large one. However, in some instances, this precipitation has been either exceeded shortly after it was published, and in others it has been considered absurdly high upon reexamination.

Besides the semantics, the actual calculation procedure for the PMP and resulting PMF tends to make several unclear assumptions and generalizations.

The most recent existing study available detailing PMP calculations in California is Hydrometeorological Report No. 59 or HMR 59 (US Army Corps of Engineers, 1999). In brief, the computational procedure includes tracing an outline of the drainage basin, placing this outline on top of a given PMP 10-mi^2, 24-hour index map, then determining depth duration relations and areal reduction factors, and finally conducting temporal distribution of incremental depths extracted from a given curve.

While this method is simple to use, and the analysis involved in creating these PMP index maps undoubtedly contains valuable information, it would better to instead adopt a probabilistic approach to precipitation analysis, where instead of assuming a deterministic, theoretical upper limit, studying existing precipitation data and extracting a return period for the already calculated 24-hour index depths, for every sub-area of the Feather River Basin, as determined by the California Department of Water Resources in (DWR, 2004). One of the possible methods to achieve this is analyzed below.

9.2 *Annual Maxima of Daily Rainfall Analysis – An alternative to the average PMP 24-hour index depth*

The 24-hour index probable maximum precipitation depth essentially describes a daily maximum precipitation value. If the distribution of daily rainfall for a given area is known, one can assume that the annual maxima of daily rainfall would resemble one of the three limiting types: type I, known as Gumbel, type II, known as Fréchet, or type III, known as reversed Weibull. As such, the Generalized Extreme Value (GEV) distribution, which comprises these types by way of its shape parameter, can be fitted to series of annual maxima of daily rainfall.

In accordance with (Koutsoyiannis, 1999); (Papalexiou & Koutsoyiannis, A probabilistic approach to the concept of Probable Maximum Precipitation, 2006); (Papalexiou & Koutsoyiannis, 2013), the GEV distribution using the method of L-moments is fitted to various precipitation data gathered from the Feather Basin.

However, constructing the input timeseries of annual daily maxima from the available daily precipitation data is not as simple as it sounds. As the daily maximum precipitation is a single value for each year, the resulting time series of annual maxima is highly sensitive. If there are no data recorded for the most intense precipitation event of a given year, the daily maximum of that specific year would be lower than the true value. Furthermore, any possible bad data has a severely negative impact on the creation of the input timeseries. For example, in several analyzed stations there exist series of days with no recorded data, with intermittent extreme values of over than 500 mm (19.69 inches) in between the blanks. Are these values actually recorded measurements, or false flags?

In the end, after applying a filter similar to that used for the creation of annual total precipitation time series in a previous chapter (i.e. only years with 300 or more daily measurements are taken into account) and discarding stations with data suspected of containing erroneous measeurements that couldn't be cross-referenced with floods around the same time period, four precipitation measurement stations (USC00044812, USC00041159, QCY, and BRS see Table 6) were selected for this analysis. Then, annual daily maxima time series were created using MATLAB, and the GEV-max distribution with the method of L-moments was fitted using the "Pythia" statistical tool of the HYDROGNOMON software. Graphs of the resulting distribution fitting can be found below.

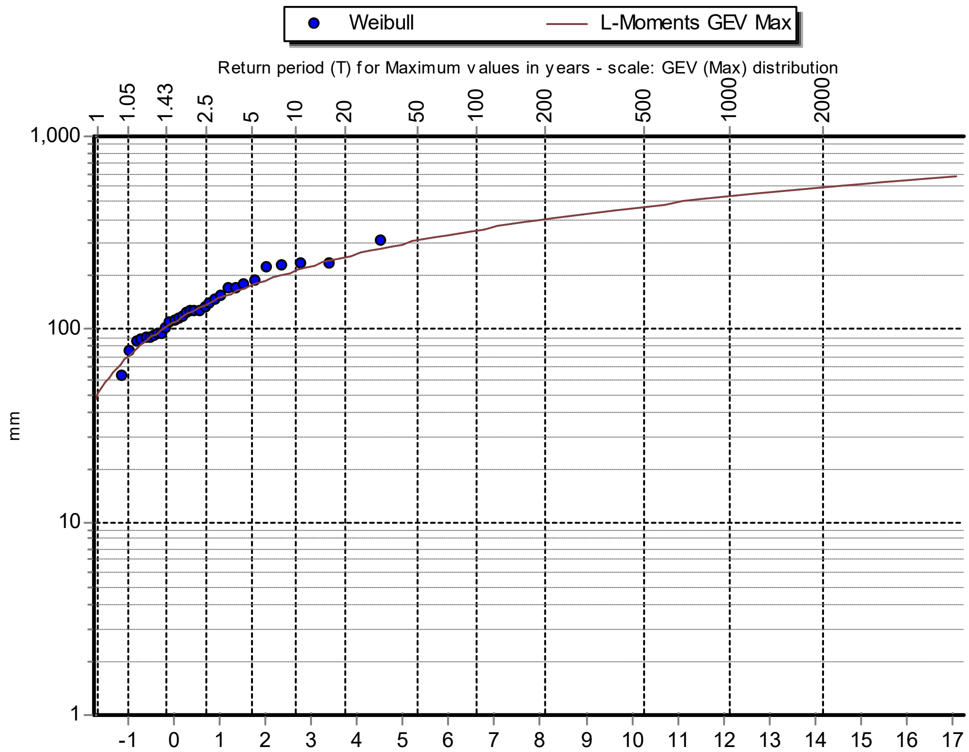


Figure 44. L-Moments GEV-Max distribution fit to annual daily maxima of precipitation measurements, Brush Creek station (BRS)

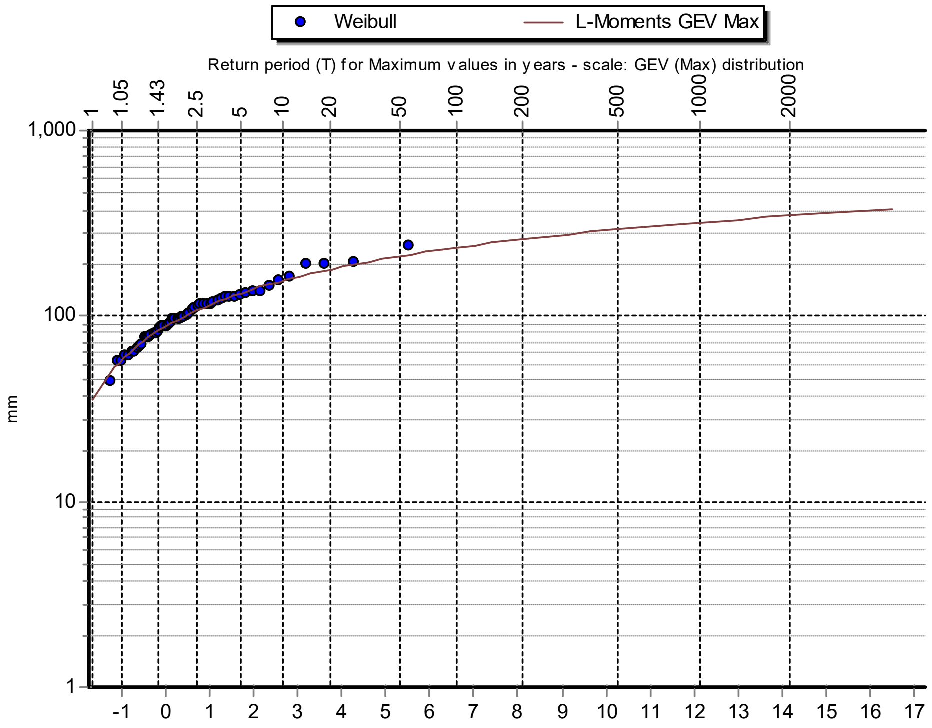


Figure 45. L-Moments GEV-Max distribution fit to annual daily maxima of precipitation measurements, station USC00044812.

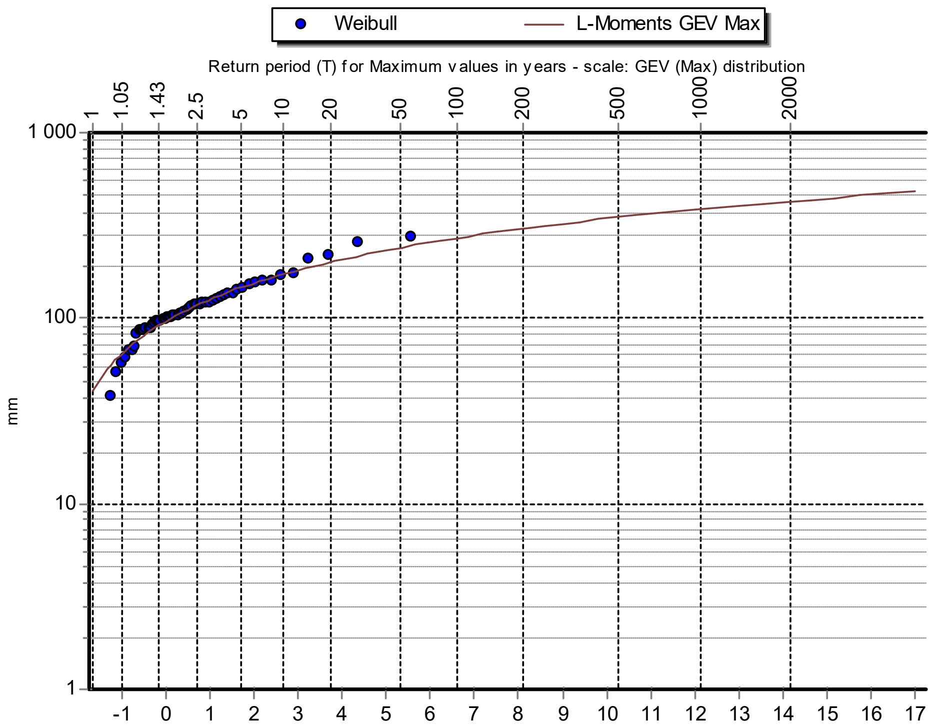


Figure 46. L-Moments GEV-Max distribution fit to annual daily maxima of precipitation measurements, station USC00041159.

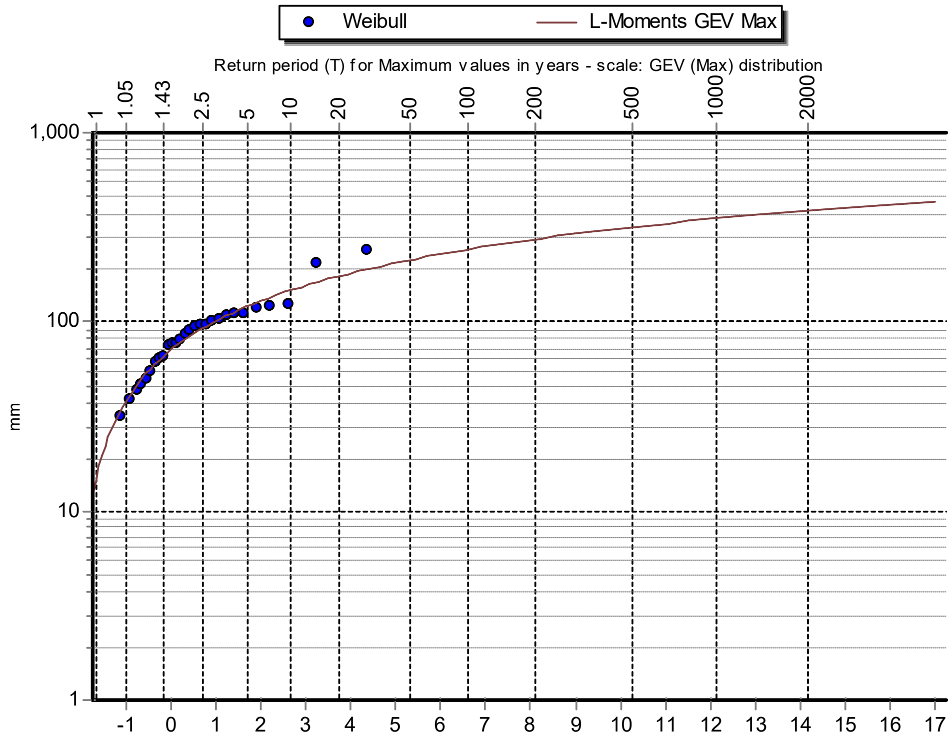


Figure 47. L-Moments GEV-Max distribution fit to annual daily maxima of precipitation measurements, Quincy station (QCY).

After consulting the 24-hour PMP index depth maps in (US Army Corps of Engineers, 1999), and comparing them to those specified in (DWR, 2004) for the subareas of the Feather River Basin, it is possible to use these distribution fits to estimate the annual daily maximum precipitation value with a 10,000 year recurrence interval, and find the return period of the stated probable maximum precipitation index depths. Below is a table summarizing the results of this analysis.

Table 17. 10,000-year recurrence interval annual daily maximum precipation forecasts, compared to the 24-hour PMP index depths and their recurrence intervals, based on the GEV-Max distribution fit.

Station ID	Available Daily Record	10,000 yr Daily Maximum Precipitation	HMR 59 Avg. PMP 24h index depth	GEV-Max Return Period of PMP
	(years)	(mm)	(mm)	(years)
BRS	1986-2017	688.6	800.1	33,333
USC00041159	1959-2016	529.7	647.7	50,000
USC00044812	1913-1967	414.8	635.0	>100,000
QCY	1989-2017	481.1	431.8	4,348

As is evident from the analysis, the PMP usually has a recurrence value that is abnormally high, which while increases safety, does tend to go beyond engineering design practices. Extreme care must also be taken to not assume that designing with the PMP method removes risk entirely simply because it generates large values. This is why assigning a return period to a design precipation value is better for understanding the probabilistic method that led to it and the risks that selecting it entails in engineering.

Futhermore, the PMP method evidently does not always generate overly extreme values. In the case of the Quincy station, the annual maxima distribution fit results in a daily maximum precipitation value with a 10,000 year return period that is slightly above the PMP 24-hour index depth for the same region. That same probable maximum value has a corresponding return period of only 4,348 years, which while is still very high, leads to the conclusion that the PMP method is not always as risk-free as some would expect.

However, these results could be negatively affected by the sensitivity of the input data time series. For this reason, Appendix D contains the annual daily maxima series used as input for the distribution fit, to promote further research and allow for cross-examination.

9.3 Flood Frequency Analysis – An alternative to the PMF

The concept of the Probable Maximum Flood is also highly controversial, for much of the same reasons as the PMP. Indeed, the fact that over the years various PMF studies for Lake Oroville have found largely varying values of probable maximum inflow and outflow does indicate that a true mathematical upper flood limit does not exist. Therefore, even the PMF is the product of a probabilistic method and designing with it in mind always will have a certain degree of risk, however small. Especially due to the extent of the Feather River Basin and the large number of smaller reservoirs within it above Oroville Dam, it is difficult to generate a true design flood without taking multiple factors into account. At the very least, it is possible to assign a return period to existing design floods by using the already familiar flood frequency analysis method.

The record of unregulated, annual maximum flow data for the Feather River at Oroville station resulting from rainfall for a 1-day duration provided by (Lamontagne, et al., 2012) is an ideal input time series for this purpose, and further cross-examination with known extreme floods such as the 1964, 1986, and 1997 events as mentioned above confirms its accuracy. Using Microsoeft Excel and HYDROGNOMON, two distributions are fitted to the data, namely the Log-Pearson III with the method of maximum likelihood estimators and the GEV distribution using the L-Moments method, according to (Interagency Advisory Committee on Water Data, 1982); (Seckin, Haktanir, & Yurtal, 2011); (Papalexiou, Koutsoyiannis, & Makropoulos, 2013). The results of the distribution fitting can be found below.

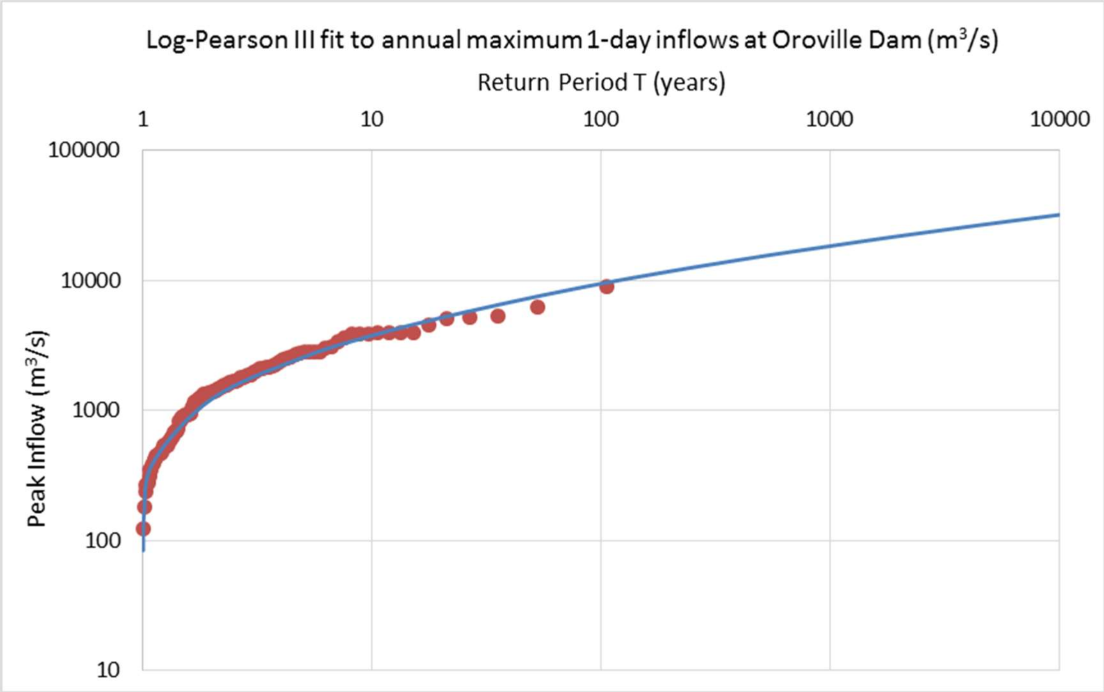


Figure 48. Log-Pearson III distribution fit to annual unregulated maximum 1-day inflows at Oroville Dam (m^3/s).

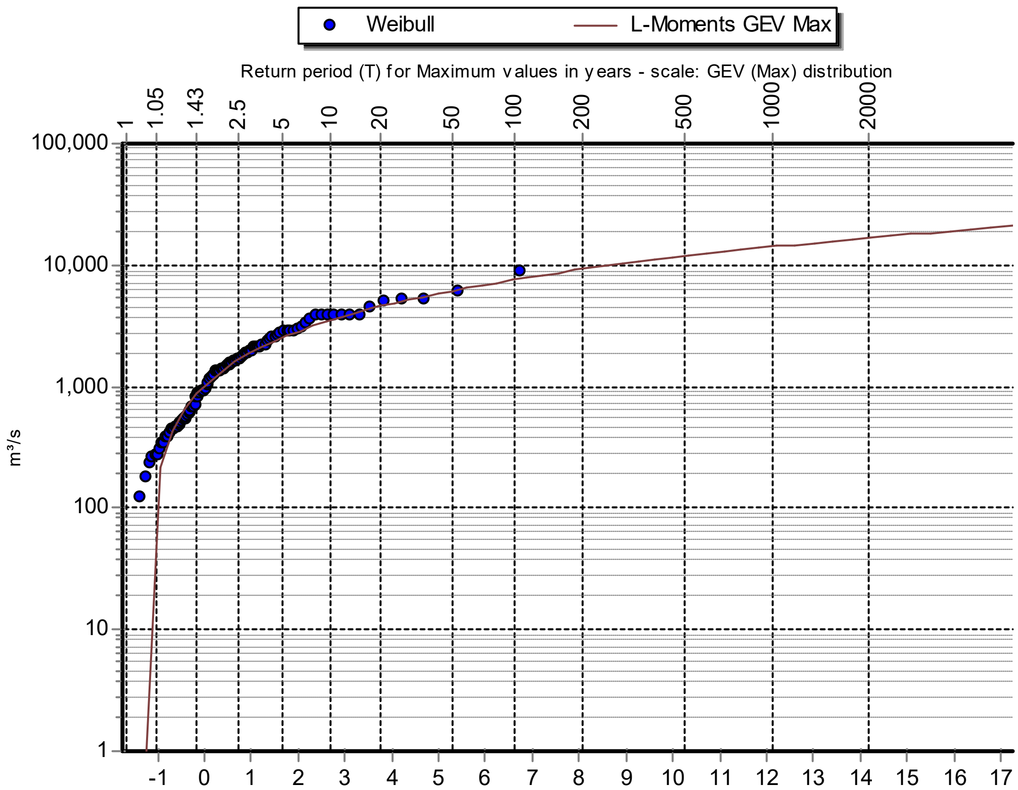


Figure 49. L-Moments GEV-Max distribution fit to annual unregulated maximum 1-day inflows at Oroville Dam (m^3/s).

From this analysis, it is possible to extract the 10,000 year floods for each of the distribution fits. For the Log-Pearson III fit, the 10,000 year flood is estimated to be 32,000 m^3/s (1,129,000 cfs) and for the GEV fit, the same value is 24,464 m^3/s (864,000 cfs). Furthermore, it is possible to assign recurrence intervals to existing calculated inflows such as the Standard Project Flood and various PMFs that can be found in (California Deparment of Water Resources, 1974); (US Army Corps of Engineers, 1970); (DWR, 2004).

Table 18. Return periods in years for various floods, as generated by the distribution fitting process.

	Peak Inflow		**Return Period (years)**	
	(cfs)	*(m^3/s)*	**LP3 fit**	**GEV fit**
1986 Flood	266,450	6,145	50	97
1997 Flood	302,013	8,860	75	150
2017 Flood	190,435	5,392	20	33
Standard Project Flood	440,000	12,459	250	610
PMF 1965	720,000	20,388	1,360	4,500
PMF 1983	1,167,000	33,046	>10,000	33,300
PMF 2003 (HMR 36)	890,000	25,202	3,500	11,100
PMF 2003 (HMR 59)	725,000	20,530	1,500	4,800

The Standard Project Flood is mentioned to have a recurrence interval of 450 years (California Deparment of Water Resources, 1974), which is close to the average of the two distribution fitting results. However, the return period of the probable maximum flood is supposed to exceed 10,000 years, yet only the 1983 PMF achieved this for both distribution fits. Notably, the most current PMF was calculated in 2003 based on HMR 59 (Interagency Advisory Committee on Water Data, 1982); (California Department of Water Resources, 2017), and its recurrence interval does not exceed 5,000 years for both distributions.

Futhermore, according to this analysis, the return period of the 2017 flood is only 20 years for the LP3 fit, and 33 years for the GEV fit. It should be noted that these flood figures are overall peaks, whereas the input for the fit are the slightly lower daily averages given by (Lamontagne, et al., 2012), so these estimates are on the conservative side. In any case, these periods should be viewed more as guidelines and not exact calculations, lest one be accused of theological speculation (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006).

9.4 *Revisiting the Minimum Flood Control Elevation*

In Chapter 8.3, it was mentioned that one of the main reasons the February 2017 storm had such a devastating impact on Oroville Dam was the fact that reservoir surface elevations were higher than those of previous significant flood events. While levels were within the flood control manual standards (US Army Corps of Engineers, 1970), the fact that they were close to the limit made dealing with the February 2017 inflows a much more daunting task once the main spillway failed. Thus, it would seem reasonable to request a lowering of the minimum flood control elevation level for Lake Oroville.

However, this is not as simple as it sounds. It is not feasible to request an arbitrary minimum flood control elevation level because it is bound by physical constraints; namely, the flood control outlet sill elevation is at 248 m (813.6 ft.). And even if one were to set that as the new minimum elevation by permanently leaving the flood gates open, the spillway would operate extremely inefficiently, as any flows that topped this elevation would simply spill down into the chute, without any kind of regulation. Outflows can also be routed through the river valve outlet and Hyatt Powerplant tailrace channels, but can output only a fraction of the spillway's discharges. Aside from that, Oroville Dam is not only designed to stop floods; it has a a number of other uses that make its ability to store water paramount to the sustainability of the Feather Basin and its downstream areas. Thus, it would be a terrible mistake to request a significant lowering of the flood control elevation without first taking into account economical and ecological factors together with flood risk management. Furthermore, since Oroville Dam's main spillway is being rebuilt, it makes sense to make as much use of this new structure as possible.

Therefore, taking all of the above into account, it would seem logical to request a small reduction in the minimum flood control level. In their 2006 statement (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006), FOR et. al, had requested an additional 150,000 acre-feet of surcharge storage be added to the 750,000 acre-feet flood control pool in order to compensate for the never constructed Marysville Dam. This was a project that was factored in the flood control pool calculations, yet was never completed. If this measure were to be implemented, according to the flood control manual, the new minimum flood control elevation is **255 m (837 ft.).** Under these conditions, according to (US Bureau of Reclamation, 1965) the flood control outlet's release capacity is approximately 1,274 m^3/s (45,000 cfs). By chance, this was the Oroville Dam resrvoir's surface elevation just before the 1997 flood (see Figure 39), and the spillway performed adequately even when outflows briefly exceeded the designed discharges. Lowering the level beyond this point would result in inefficient outflows from the spillway, so this is considered to be the "sweet spot" for the Oroville Dam reservoir.

10. Conclusions and Moving Forward

The Oroville Dam 2017 spillway incident will be remembered in history for its uniqueness, as it is a failure of a dam's key structure that occurred under standard operating conditions, yet at an unforunate time. It makes for a very interesting problem from a dam operator's perspective; what does one do when a spillway, a structure built to deal for emergency situations, fails just when it is needed? And in the specific case of Oroville Dam, is the auxiliary spillway a feature, or a mark of a critical flaw in its design? While it would indeed save the main dam from overtopping in the event of a probable maximum flood, in doing so it would likely not be able to hold for long, failing and releasing 10 m (30 ft.) of the reservoir's water downstream, flooding an enormous area with more than 180,000 permanent residents. Furthermore, this aforementioned probable maximum flood seems more probable then presumed, and it's definitely not a maximum.

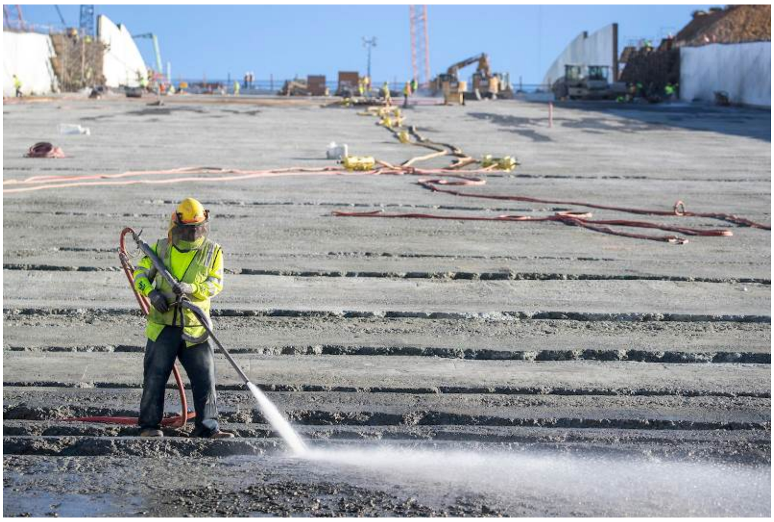


Image 41. October 25, 2017. A worker uses water and compressed air to clear the concrete floor of the new Oroville Dam spillway chute, in preparation for a new layer of RCC. The new structure is nearly complete. Source: Ken James, DWR.

An independent forensic team tasked with determining the causes of the spillway incident recently published a summary of their findings (Oroville Dam Spillway Incident Independent Forensic Team, 2017). With the ability to conduct an on-site investigation, they were able to confirm some of the causes mentioned in this thesis as well as outline new ones. Namely, the redesign of chute's underdrain system apparently led to an inconsistent thickness in the concrete floor slabs, which resulted in cracks above the herringbone drains, allowing water to pass through the slabs and also potentially led to concrete spalling. Futhermore, the anchorage of the concrete to the foundation was in some places developed in weathered sections of rock, leading to a pullout strength lower than the intended design.

The intent of this thesis is to review the causes that led to the Oroville Dam spillway incident and see how they can be avoided, in order to avoid similar events in the future. Thus, based on the above analysis and after consulting dam inspection manuals (Morrison-Knudsen Engineers, Inc., 1986) and reviewing the on-site investigation report (Oroville Dam Spillway Incident Independent Forensic Team, 2017), the following conclusions are drawn:

- From a structural standpoint, the main spillway chute appears to have initially failed due to uplift of its concrete floor slabs, caused somewhere between Stations +33 00 and +33 50 (2,000 and 2,050 feet of its rectangular section length, respectively). This uplift appears to have been caused by water accumulating below the chute floor, which was unable to be routed through the underdrains. This evidenced by photographs showing them operating under pressure, which should never occur under design specifications.
- The rest of the damage to the main spillway was caused by high velocity flows due to the large amount of water that had to be routed through it to avoid erosion downstream of the emergency spillway.
- The fact that Lake Oroville's surface elevation was at the minimum flood control level, above that during previous major flood events, resulted in more severe conditions, even though the February 2017 inflows were not record-high. Thus, a lowering of the minimum flood control level to **255 m (837 ft.)** is recommended. (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006) reveal that this actually would not be a new requirement, but an adaptation to outdated assumptions made in the 1970 flood control manual (US Army Corps of Engineers, 1970). Based on the main spillway rating curve (US Bureau of Reclamation, 1965), it would be feasible to maintain the dam reservoir at this level during wet seasons.
- The current PMF for Lake Oroville has a return period of less than 10,000 years based on the above analysis. It is recommended to either calculate a new 10,000 year flood for Lake Oroville using a probabilistic method, or use the 1983 PMF which is suitably large. However, the state must assign a recurrence interval to any resulting flood, as the term "probable maximum flood" is outdated (Koutsoyiannis, 1999).
- The California Department of Water Resources' quick response to the incident and initiation of a full scale repair and reconstruction of the Oroville Dam spillways is greatly appreciated. However, under current design, the dam is only capable of withstanding the Standard Project Flood with a return period of 500 years without sustaining significant damage. The emergency spillway should be immediately redesigned to be fully armored with concrete in order to withstand a flood with a recurrence interval of 10,000 years without causing significant erosions to the downstream areas. This has been repeatedly requested by local interest groups (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006) (Schnoover, 2017).

In the United States, many are using this incident as a textbook example of severe issues the country has with maintaining the gigantic number of high-risk structures it has built over the past century (BBC, 2017); (Stork, et al., 2017). Indeed, Oroville Dam itself has reached the halfway point of its expected life as a structure. Until a major problem occurs at a critical facility like this one, it is easy to get complacent and avoid or postpone critical maintenance procedures like routine inspections and small repairs. And even when larger problems or design flaws are pointed out (Friends of the River; Sierra Club; South Yuba River Citizens League, 2006), it is difficult to convince the authorities to fund large-scale repair projects. However, one would argue that such repair projects actually conserve money in the long run. The new Oroville Dam spillway is estimated to cost around $500 million (Rogers, 2017) which is significantly more than what would have been required for a full concrete armoring of the emergency spillway back in 2006. And this is without taking into account the lives and properties of the downstream community, who deserve to live in safety.

After the incident, the California Department of Water Resources seems to have taken a different stand on the issue, being more open to suggestions about the construction of the new spillways (California Department of Water Resources, 2017). Still, this response has comes at a rather late time, and is being met with some criticism (Schnoover, 2017); (Stork, et al., 2017). However, their stance on providing free access data to the public, and attempting to communicate and cooperate with local residents and interest groups is, while not exemplary, definitely a step in the right direction. It must be stated that this work would not be nearly as complete as it is without the large amount of digital information available directly from the Department of Water Resources and related websites.

If there is a lesson that must be learned from this incident, it is that even when a critical, yet aging structure like Oroville Dam seems to operate up to standard, one small flaw can emerge at any time and result in a severe failure due to the sheer scale of the facilities and the conditions they are expected to consistently work under. While routine official inspections by the dam operators and independent authorities are a necessity, they are simply not enough as time goes by. Informal inspections of all related facilities must be conducted by dam operators on a weekly or bi-weekly basis, in accordance with existing guidelines (Morrison-Knudsen Engineers, Inc., 1986), not with the intent of writing official reports, but simply to detect the telltale signs of imminent failure before the theoretical worst outcome becomes a reality. If the dam operators had noticed the differences in the main spillway chute’s floor slabs between mid and late January they might have been able to repair it in time and avoid the incident from occurring entirely, or at least mitigate its results.

Furthermore, this incident shows a possible lack of regulatory requirements based around the prevention of failures that could occur during normal operating conditions such what happened at Oroville Dam. These incidents are important as well, as even though no lives were lost as a result of the incident, its consequences on the local environment, economy, and communities will be felt in the years to come.

In the end, while it takes a great amount of knowledge, research, and responsibility to build a large dam, it takes much more to consistently operate one and protect it from damage. It is a thankless task, as when maintenance is done right, nothing happens. Yet someone has to do it.

11. Suggestions for Future Research

The Oroville Dam 2017 spillway incident is a multi-faceted topic of research, covering nearly all the aspects of modern day civil and environmental engineering and providing a basis for multiple future projects. Proposals for further research can be found below.

- (Moustakis, 2017) has developed a pseudo-continuous stochastic simulation framework with the purpose of estimating flood flows. However, it is very recent and has only been investigated on a theoretical level, and hasn't yet been tested upon actual flood data. The available time series for Oroville should be more than enough to validate the predictive capabilities of this framework.
- The exisiting flood inundation maps for areas downstream of Oroville Dam need to be updated, to correspond with a 10,000 year flood generated from a probabilistic method. However, it is not necessary to create an inundation map for the 10,000 year flood itself, generating floodplain boundaries for the 500-year and 1,000 year events as seen in (DWR, 2004) is enough.
- One of the greatest consequences of the spillway incident were those to the environment, yet this was not covered within the scope of this thesis. Further research could yield a full assessment of the environmental impact of Oroville Dam on the Feather Basin ecosystem before and after the incident.
- In a similar vein, the long-term socio-economic consequences of the spillway incident on the downstream communities were also not analyzed extensively, yet remain a very important aspect of research. Most of the available data consists of news reports that only cover the short-term consequences of the evacuation, yet this factor deserves research that goes on a deeper level.
- A proper precipitation-runoff model of the Feather Basin needs to be constructed. Papers already detail how this could be accomplished (Koczot, Jeton, McGurk, & Dettinger, 2005). The large number of small reservoirs within the basin and the multiple factors that need to be taken into account for this model make it a very complicated, yet also interesting project.

- At the time of the incident, many news outlets were blaming a long drought period in California as the cause of a possible mis-management of the Oroville Dam reservoir. Critics mention that the reasons for keeping the reservoir surface elevation near the minimum flood control level and not safely below it could be linked to a possible expectation of yet another dry year followed by future increased water demands in the summer (Hagen, 2017). Thus, a need arises to examine whether California's droughts are a new phenomenon, produced by climate change, or if extreme dry years always have and always will be followed by extreme wet years, which would be an indication of climate persistence.
- According to multiple news reports, The Oroville Dam incident is yet another example of failing infrastructure in the United States. While this topic has already been extensively covered, until a proper plan for solving this issue is put forward, it remains a relevant topic for conducting research.
- The IFT interim memo (Oroville Dam Spillway Incident Independent Forensic Team, 2017) states that inspection methods need to adapted to contain procedures that might detect operational or design mistakes that would impact a dam operating under normal conditions. This is a key part of why the Oroville Dam spillway incident became such a huge issue out of seemingly nowhere, and developing an inspection procedure that would discover these small flaws reliably is a very complicated task, which requires extensive research of this and other incidents on a global scale.

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Appendix A – Annual Hydrologic Data of Analyzed Stations in the Feather River Basin

Table A-1. Annual total precipitation for the years 1913 to 1967, station USC00044812.

Year	Total Precipitation (mm)	Total Precipitation (inches)
1913	768.3	30.2
1914	367.7	14.5
1915	1623.2	63.9
1916	1418.6	55.9
1917	796.8	31.4
1918	783.1	30.8
1919	897.8	35.3
1920	1267.2	49.9
1921	1101.3	43.4
1922	1285.7	50.6
1923	545.6	21.5
1924	841.8	33.1
1925	837.5	33.0
1926	1317.7	51.9
1927	1438.7	56.6
1928	1014.4	39.9
1929	968.7	38.1
1930	792.2	31.2
1931	1096.4	43.2
1932	621.1	24.5
1933	1056.1	41.6
1934	880.4	34.7
1935	1106.2	43.6
1936	1399.7	55.1
1937	1807.1	71.1
1938	1484.1	58.4
1939	678.1	26.7
1940	2256.1	88.8
1941	2011.7	79.2
1942	1520.9	59.9
1943	1098.4	43.2
1944	1321.4	52.0
1945	1483.0	58.4
1946	672.3	26.5
1947	882.4	34.7
1948	1299.5	51.2
1949	735.5	29.0
1950	1787.1	70.4
1951	1438.1	56.6
1952	1587.5	62.5
1953	1102.0	43.4
1954	1431.4	56.4
1955	1524.3	60.0

1956	1079.5	42.5
1957	1382.9	54.4
1958	1454.8	57.3
1959	867.4	34.1
1960	1307.2	51.5
1961	940.3	37.0
1962	1696.9	66.8
1963	1059.0	41.7
1964	1354.5	53.3
1965	1279.0	50.4
1966	641.7	25.3
1967	1127.0	44.4
Average	1173.4	46.2
St. Deviation	384.7	15.1
Skewness Coefficient	0.4	0.4
Excess Kurtosis Coefficient	0.1	0.1

Table A-2. Annual total precipitation for the years 1959 to 2016, station USC00041159.

Year	Total Precipitation (mm)	Total Precipitation (inches)
1959	258.5	10.2
1960	2216.1	87.2
1961	1429.1	56.3
1962	2041.3	80.4
1963	1765.1	69.5
1964	1996	78.6
1965	1578	62.1
1966	1254.4	49.4
1967	1773.5	69.8
1968	1673.8	65.9
1969	2072.5	81.6
1970	2280.7	89.8
1971	1302.5	51.3
1972	1354.5	53.3
1973	2374.1	93.5
1974	1665.7	65.6
1975	1672.4	65.8
1976	550.5	21.7
1977	930.1	36.6
1978	2008.4	79.1
1979	1944	76.5
1980	1719.5	67.7
1981	2112.4	83.2
1982	2137.5	84.2
1983	3428.3	135.0
1984	1347.8	53.1
1985	1078.1	42.4
1986	1464.2	57.6
1987	1342.4	52.9
1988	1247.2	49.1
1989	1545.1	60.8
1990	1019.9	40.2
1991	1327.6	52.3
1992	1793.7	70.6
1993	1863	73.3
1994	1242.1	48.9
1995	3160.1	124.4
1996	2826.4	111.3
1997	854.3	33.6
1998	2966.5	116.8
1999	1159.4	45.6
2000	-	-
2001	992.4	39.1
2002	1546.7	60.9
2003	1501.1	59.1
2004	1168.7	46.0
2005	1966	77.4
2006	1931.7	76.1
2007	990.7	39.0
2008	1190.7	46.9
2009	1347.5	53.1

2010	2055.7	80.9
2011	1429.8	56.3
2012	2020.8	79.6
2013	366	14.4
2014	1435.6	56.5
2015	855.2	33.7
2016	89.4	3.5
Average	1590.6	62.6
St. Deviation	653.1	25.7
Skewness Coefficient	0.4	0.4
Excess Kurtosis Coefficient	0.9	0.9

Table A-3. Annual total precipitation for the years 1905 to 1979, station QCY.

Year	Precipitation (mm)	Precipitation (inches)
1905	751.08	29.57
1906	1699.77	66.92
1907	1706.63	67.19
1908	822.20	32.37
1909	1749.30	68.87
1910	600.71	23.65
1911	1192.28	46.94
1912	630.43	24.82
1913	841.76	33.14
1914	1027.43	40.45
1915	739.90	29.13
1916	1479.55	58.25
1917	653.29	25.72
1918	715.77	28.18
1919	906.27	35.68
1920	1121.66	44.16
1921	922.78	36.33
1922	1233.42	48.56
1923	573.79	22.59
1924	624.33	24.58
1925	873.00	34.37
1926	1247.90	49.13
1927	1135.13	44.69
1928	832.61	32.78
1929	760.98	29.96
1930	730.50	28.76
1931	882.90	34.76
1932	524.26	20.64
1933	738.12	29.06
1934	704.09	27.72
1935	883.16	34.77
1936	897.13	35.32
1937	1343.15	52.88
1938	1121.16	44.14
1939	598.17	23.55
1940	1748.28	68.83
1941	1205.23	47.45
1942	1279.40	50.37
1943	931.67	36.68
1944	972.31	38.28
1945	1221.49	48.09
1946	640.84	25.23
1947	620.78	24.44

1948	1112.77	43.81
1949	562.61	22.15
1950	1688.59	66.48
1951	1306.83	51.45
1952	1209.29	47.61
1953	932.18	36.70
1954	1157.73	45.58
1955	1169.16	46.03
1956	952.75	37.51
1957	1138.94	44.84
1958	1187.45	46.75
1959	743.71	29.28
1960	1003.30	39.50
1961	693.67	27.31
1962	1294.64	50.97
1963	1130.05	44.49
1964	1086.61	42.78
1965	1053.08	41.46
1966	850.39	33.48
1967	1187.20	46.74
1968	1090.17	42.92
1969	1436.12	56.54
1970	1503.68	59.20
1971	959.87	37.79
1972	768.86	30.27
1973	1410.21	55.52
1974	1130.05	44.49
1975	912.62	35.93
1976	275.59	10.85
1977	528.32	20.80
1978	1252.47	49.31
1979	534.42	21.04
Average	1001.97	39.45
St. Deviation	326.17	12.84
Skewness Coefficient	0.40	0.40
Excess Kurtosis Coefficient	-0.19	-0.19

Table A-4. Annual total precipitation for the years 1907 to 1982, station CNY.

Year	Precipitation (mm)	Precipitation (inches)
1907	262.13	10.32
1908	582.17	22.92
1909	1256.28	49.46
1910	741.93	29.21
1911	1100.33	43.32
1912	696.21	27.41
1913	915.92	36.06
1914	979.42	38.56
1915	1114.30	43.87
1916	1073.66	42.27
1917	768.35	30.25
1918	788.67	31.05
1919	808.74	31.84
1920	992.38	39.07
1921	874.01	34.41
1922	1078.23	42.45
1923	423.67	16.68
1924	533.40	21.00
1925	825.25	32.49
1926	1002.28	39.46
1927	990.09	38.98
1928	701.55	27.62
1929	759.21	29.89
1930	636.27	25.05
1931	914.65	36.01
1932	550.16	21.66
1933	799.08	31.46
1934	689.86	27.16
1935	1005.33	39.58
1936	945.39	37.22
1937	1297.94	51.10
1938	1169.16	46.03
1939	568.45	22.38
1940	1581.66	62.27
1941	1268.98	49.96
1942	1047.75	41.25
1943	789.43	31.08
1944	945.39	37.22
1945	1049.27	41.31
1946	599.69	23.61
1947	695.96	27.40
1948	1052.83	41.45
1949	570.74	22.47

1950	1386.33	54.58
1951	1140.97	44.92
1952	1249.43	49.19
1953	891.79	35.11
1954	1041.65	41.01
1955	1052.83	41.45
1956	1054.86	41.53
1957	1123.19	44.22
1958	1092.96	43.03
1959	682.24	26.86
1960	988.31	38.91
1961	739.65	29.12
1962	1234.69	48.61
1963	1016.76	40.03
1964	1104.14	43.47
1965	1004.06	39.53
1966	802.89	31.61
1967	1160.53	45.69
1968	966.72	38.06
1969	1308.61	51.52
1970	1331.98	52.44
1971	885.44	34.86
1972	790.45	31.12
1973	1357.38	53.44
1974	1039.62	40.93
1975	1050.29	41.35
1976	366.01	14.41
1977	650.24	25.60
1978	1111.25	43.75
1979	857.25	33.75
1980	936.75	36.88
1981	1280.67	50.42
1982	604.27	23.79
Average	930.93	36.65
St. Deviation	257.04	10.12
Skewness Coefficient	-0.17	-0.17
Excess Kurtosis Coefficient	-0.09	-0.09

Table A-5. Annual total precipitation for the years 1920 to 1995, station CBO.

Year	Precipitation (mm)	Precipitation (inches)
1920	518.16	20.40
1921	754.89	29.72
1922	1070.10	42.13
1923	462.79	18.22
1924	600.20	23.63
1925	883.41	34.78
1926	1163.32	45.80
1927	1177.54	46.36
1928	837.95	32.99
1929	845.06	33.27
1930	792.73	31.21
1931	980.95	38.62
1932	563.63	22.19
1933	861.31	33.91
1934	699.26	27.53
1935	954.53	37.58
1936	985.27	38.79
1937	1425.19	56.11
1938	1196.34	47.10
1939	583.95	22.99
1940	1639.82	64.56
1941	1425.70	56.13
1942	1177.80	46.37
1943	901.19	35.48
1944	1005.08	39.57
1945	1219.45	48.01
1946	617.47	24.31
1947	653.80	25.74
1948	1140.71	44.91
1949	602.49	23.72
1950	1485.65	58.49
1951	1230.88	48.46
1952	1418.34	55.84
1953	883.92	34.80
1954	1107.69	43.61
1955	1043.43	41.08
1956	1060.70	41.76
1957	1211.58	47.70
1958	1241.04	48.86
1959	723.14	28.47
1960	1116.08	43.94
1961	748.03	29.45
1962	1277.37	50.29

1963	1045.72	41.17
1964	1107.95	43.62
1965	994.92	39.17
1966	782.32	30.80
1967	1176.53	46.32
1968	1137.16	44.77
1969	1285.75	50.62
1970	1410.21	55.52
1971	881.89	34.72
1972	887.73	34.95
1973	1416.56	55.77
1974	1105.41	43.52
1975	1062.99	41.85
1976	357.89	14.09
1977	672.85	26.49
1978	1263.40	49.74
1979	1137.67	44.79
1980	1304.54	51.36
1981	1339.60	52.74
1982	1403.10	55.24
1983	1962.66	77.27
1984	808.23	31.82
1985	669.80	26.37
1986	1282.45	50.49
1987	816.86	32.16
1988	651.00	25.63
1989	867.66	34.16
1990	650.75	25.62
1991	889.51	35.02
1992	953.01	37.52
1993	1258.06	49.53
1994	867.41	34.15
1995	1364.74	53.73
Average	1014.53	39.94
St. Deviation	300.56	11.83
Skewness Coefficient	0.26	0.26
Excess Kurtosis Coefficient	0.16	0.16

Table A-6. Annual total precipitation for the years 1935 to 2010, station BRS.

Year	Precipitation (mm)	Precipitation (inches)
1935	383.79	15.11
1936	2183.13	85.95
1937	2550.41	100.41
1938	2025.14	79.73
1939	881.89	34.72
1940	2900.93	114.21
1941	2680.21	105.52
1942	2022.35	79.62
1943	1610.87	63.42
1944	1934.97	76.18
1945	2142.49	84.35
1946	1028.95	40.51
1947	1377.44	54.23
1948	1885.44	74.23
1949	1041.15	40.99
1950	2666.49	104.98
1951	2185.16	86.03
1952	2177.80	85.74
1953	1679.70	66.13
1954	1830.07	72.05
1955	1950.21	76.78
1956	1407.16	55.40
1957	1889.00	74.37
1958	2033.02	80.04
1959	1258.57	49.55
1960	1941.58	76.44
1961	1334.26	52.53
1962	2329.43	91.71
1963	1697.48	66.83
1964	1851.91	72.91
1965	1520.44	59.86
1966	1418.34	55.84
1967	1783.33	70.21
1968	1747.52	68.80
1969	2282.44	89.86
1970	2569.46	101.16
1971	1250.19	49.22
1972	1283.46	50.53
1973	2608.07	102.68
1974	1750.06	68.90
1975	1597.15	62.88
1976	514.10	20.24
1977	1121.92	44.17

1978	2291.08	90.20
1979	1971.29	77.61
1980	1648.21	64.89
1981	2525.01	99.41
1982	2451.35	96.51
1983	3380.49	133.09
1984	1247.65	49.12
1985	1043.18	41.07
1986	1762.00	69.37
1987	1442.97	56.81
1988	1367.54	53.84
1989	1417.07	55.79
1990	1173.99	46.22
1991	1521.71	59.91
1992	1690.12	66.54
1993	2029.71	79.91
1994	1398.27	55.05
1995	2586.74	101.84
1996	2999.49	118.09
1997	1688.34	66.47
1998	2789.17	109.81
1999	1457.20	57.37
2000	1887.98	74.33
2001	1634.49	64.35
2002	1712.98	67.44
2003	1876.81	73.89
2004	1553.21	61.15
2005	2576.83	101.45
2006	2437.89	95.98
2007	1359.41	53.52
2008	1302.00	51.26
2009	1176.02	46.30
2010	1319.78	51.96
Average	1803.26	70.99
St. Deviation	575.60	22.66
Skewness Coefficient	0.25	0.25
Excess Kurtosis Coefficient	0.08	0.08

Table A-7. Annual total full natural flow, Feather River at Oroville (FTO) station.

FTO					
Water Year	**Flow (hm3)**	**Water Year**	**Flow (hm3)**	**Water Year**	**Flow (hm3)**
1905-06	8456.26	**1943-44**	3542.68	**1981-82**	11098.83
1906-07	11708.7	**1944-45**	4607.79	**1982-83**	11616.81
1907-08	4488.89	**1945-46**	5161.88	**1983-84**	7113.3
1908-09	9271.96	**1946-47**	3123.18	**1984-85**	3258.33
1909-10	5720.27	**1947-48**	4753.47	**1985-86**	8338.46
1910-11	8774.5	**1948-49**	3201.13	**1986-87**	2747.09
1911-12	2822.95	**1949-50**	4737.93	**1987-88**	2527.09
1912-13	3450.91	**1950-51**	7019.75	**1988-89**	4548.1
1913-14	8621.42	**1951-52**	9820.98	**1989-90**	2677.9
1914-15	6716.31	**1952-53**	6433.22	**1990-91**	2536.73
1915-16	7652.52	**1953-54**	5217.63	**1991-92**	2340.58
1916-17	5763.57	**1954-55**	3049.17	**1992-93**	7047.49
1917-18	3334.47	**1955-56**	9835.91	**1993-94**	2332.91
1918-19	4474.46	**1956-57**	4469.89	**1994-95**	11446.11
1919-20	2733.77	**1957-58**	8597.49	**1995-96**	7133.07
1920-21	7342.79	**1958-59**	3516.9	**1996-97**	8331.39
1921-22	6247.09	**1959-60**	3975.02	**1997-98**	8879.97
1922-23	3818.74	**1960-61**	3252.88	**1998-99**	6509.99
1923-24	1597.73	**1961-62**	4512.99	**1999-00**	5236.09
1924-25	3793.82	**1962-63**	7729.44	**2000-01**	2517.05
1925-26	3821.08	**1963-64**	3192.29	**2001-02**	3804.47
1926-27	6993.47	**1964-65**	8525.48	**2002-03**	5788.22
1927-28	5146.21	**1965-66**	3522.45	**2003-04**	4687.4
1928-29	2274.91	**1966-67**	7749.97	**2004-05**	5262.62
1929-30	4874.72	**1967-68**	4266	**2005-06**	10129.69
1930-31	1780.28	**1968-69**	8719.36	**2006-07**	3133.52
1931-32	4100.59	**1969-70**	7732.82	**2007-08**	2761.19
1932-33	2466.35	**1970-71**	7349.58	**2008-09**	3881.51
1933-34	2487.69	**1971-72**	3987.72	**2009-10**	4422.94
1934-35	5266.97	**1972-73**	5847.44	**2010-11**	8114.8
1935-36	5292.13	**1973-74**	10315.45	**2011-12**	3526.99
1936-37	3905.33	**1974-75**	5987.49	**2012-13**	3860.41
1937-38	10612.75	**1975-76**	2281.28	**2013-14**	2122.22
1938-39	2290.58	**1976-77**	1226.65	**2014-15**	2486.4
1939-40	6999.76	**1977-78**	7012.59	**2015-16**	5240.72
1940-41	7995.43	**1978-79**	3728.41	**2016-17**	12556.63
1941-42	8205.12	**1979-80**	6824.74		
1942-43	6932.17	**1980-81**	3056.11		

Table A-8. Annual total full natural flow, Feather Middle Fork near Merrimac (FTM) station.

FTM					
Water Year	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**
1907-08	912.65	**1931-32**	1009.69	**1955-56**	2219.91
1908-09	1909	**1932-33**	462.72	**1956-57**	977.94
1909-10	1177.86	**1933-34**	435.14	**1957-58**	1901.97
1910-11	1948.68	**1934-35**	1275.53	**1958-59**	628.85
1911-12	508.86	**1935-36**	1187.16	**1959-60**	823.1
1912-13	728.38	**1936-37**	860.59	**1960-61**	555.28
1913-14	2065.01	**1937-38**	2567.27	**1961-62**	940.63
1914-15	1466.45	**1938-39**	382.96	**1962-63**	1673.43
1915-16	1798.5	**1939-40**	1574.37	**1963-64**	651.93
1916-17	1367.36	**1940-41**	1680.16	**1964-65**	1992.2
1917-18	674.3	**1941-42**	1747.87	**1965-66**	714.79
1918-19	1014.24	**1942-43**	1548.46	**1966-67**	1788.18
1919-20	574.27	**1943-44**	670.51	**1967-68**	821.62
1920-21	1561.83	**1944-45**	1024.04	**1968-69**	1919.22
1921-22	1576.65	**1945-46**	1118.45	**1969-70**	1539.79
1922-23	889.9	**1946-47**	588.48		
1923-24	193.22	**1947-48**	1044.62		
1924-25	813.88	**1948-49**	691.86		
1925-26	774.87	**1949-50**	1107.3		
1926-27	1692.86	**1950-51**	1562.94		
1927-28	1151.31	**1951-52**	2467.54		
1928-29	416.62	**1952-53**	1341.12		
1929-30	1076.23	**1953-54**	999.28		
1930-31	268.84	**1954-55**	569.89		

Table A-9. Annual total full natural flow, Feather North Fork at Pulga (FPL) station.

FPL					
Water Year	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**
1911-12	1691.1	**1939-40**	3701.06	**1967-68**	2388.88
1912-13	1893.39	**1940-41**	4236.39	**1968-69**	4462.86
1913-14	4244.04	**1941-42**	4369.36	**1969-70**	3821.57
1914-15	3198.54	**1942-43**	3694.28	**1970-71**	3874.08
1915-16	3823.67	**1943-44**	2090.14	**1971-72**	2237.08
1916-17	3120.22	**1944-45**	2419.84	**1972-73**	2806.27
1917-18	1981.1	**1945-46**	2796.55	**1973-74**	5181.73
1918-19	2411.58	**1946-47**	1793.73	**1974-75**	3256.81
1919-20	1637.57	**1947-48**	2620.41	**1975-76**	1323.55
1920-21	3454	**1948-49**	1798.42	**1976-77**	821.61
1921-22	3178.93	**1949-50**	2518.65	**1977-78**	3336.67
1922-23	2104.69	**1950-51**	3483.85	**1978-79**	1794.28
1923-24	1009.97	**1951-52**	4985.61	**1979-80**	3177.75
1924-25	1942.24	**1952-53**	3564.52	**1980-81**	1713.34
1925-26	2131.58	**1953-54**	2860.2	**1981-82**	4762.27
1926-27	3657.15	**1954-55**	1766.22	**1982-83**	5703.84
1927-28	2656.3	**1955-56**	5006.46	**1983-84**	3407.01
1928-29	1292.2	**1956-57**	2447.35	**1984-85**	1800.99
1929-30	2563.92	**1957-58**	4511.21	**1985-86**	4127.67
1930-31	1122.47	**1958-59**	2099.76	**1986-87**	1604.24
1931-32	2257.27	**1959-60**	2167.54	**1987-88**	1378.14
1932-33	1508.05	**1960-61**	1879.39	**1988-89**	2313.73
1933-34	1462.42	**1961-62**	2358.24	**1989-90**	1644.07
1934-35	2536.78	**1962-63**	4157.65	**1990-91**	1426.74
1935-36	2778.66	**1963-64**	1790.5	**1991-92**	1443.45
1936-37	2105.68	**1964-65**	4348.5	**1992-93**	3579.81
1937-38	5427.32	**1965-66**	2025.87	**1993-94**	1525.4
1938-39	1245.82	**1966-67**	4174.97		

Table A-10. Annual total full natural flow, Feather South Fork at Ponderosa (FTP) station.

FTP					
Water Year	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**	**Water Year**	**Flow (hm^3)**
1900-01	506.61	**1931-32**	267.28	**1962-63**	436.43
1901-02	375.1	**1932-33**	131	**1963-64**	168.62
1902-03	384.46	**1933-34**	149.62	**1964-65**	535.3
1903-04	679.88	**1934-35**	310.47	**1965-66**	183.99
1904-05	392.62	**1935-36**	352.28	**1966-67**	421.24
1905-06	537.05	**1936-37**	247.94	**1967-68**	222.25
1906-07	693.21	**1937-38**	637.46	**1968-69**	657.56
1907-08	319.72	**1938-39**	124.72	**1969-70**	364.75
1908-09	562.59	**1939-40**	421.28	**1970-71**	399.91
1909-10	390.39	**1940-41**	524.22	**1971-72**	359.1
1910-11	543.99	**1941-42**	476.62	**1972-73**	616.46
1911-12	167.5	**1942-43**	403.63	**1973-74**	660.17
1912-13	219.18	**1943-44**	186.66	**1974-75**	480.52
1913-14	549.39	**1944-45**	294.39	**1975-76**	92.27
1914-15	463.92	**1945-46**	314.34	**1976-77**	39.25
1915-16	476.85	**1946-47**	173.83	**1977-78**	610.39
1916-17	353.63	**1947-48**	287.13	**1978-79**	459.75
1917-18	174.89	**1948-49**	190.16	**1979-80**	446.87
1918-19	267.66	**1949-50**	285.03	**1980-81**	165.72
1919-20	202.77	**1950-51**	474.72	**1981-82**	700.29
1920-21	510.54	**1951-52**	587.74	**1982-83**	708.48
1921-22	380.66	**1952-53**	398.2	**1983-84**	400.8
1922-23	241.39	**1953-54**	315.98	**1984-85**	172.06
1923-24	82.63	**1954-55**	174.26	**1985-86**	497.34
1924-25	293.81	**1955-56**	599.84	**1986-87**	144.94
1925-26	209.58	**1956-57**	273.5	**1987-88**	155.77
1926-27	442.09	**1957-58**	534.26	**1988-89**	
1927-28	320.33	**1958-59**	197.49	**1989-90**	146.84
1928-29	136.17	**1959-60**	251.67	**1990-91**	
1929-30	287.19	**1960-61**	184.07	**1991-92**	173.92
1930-31	87.96	**1961-62**	279.08		

Table A-11. Annual unregulated maximum flood events resulting from rainfall for n-day durations, Feather River at Oroville. Sources: USACE, (Lamontagne, et al., 2012)

	1-day		3-day		7-day		15-day		30-day	
Water Year	**Date**	**Flow (m^3/s)**	**Date**	**Flow (m^3/s)**	**Date**	**Flow (m^3/s)**	**Date**	**Flow (m^3/s)**	**Date**	**Flow (m^3/s)**
1901-02	7-Apr	1078.87	5-Apr	1012.89	24-Feb	826.57	15-Feb	659.22	7-Feb	538.02
1902-03	30-Mar	2633.47	30-Mar	1878.26	30-Mar	1263.21	30-Mar	862.81	14-Mar	546.23
1903-04	24-Feb	3001.59	18-Mar	2501.23	22-Feb	1880.52	16-Feb	1408.48	22-Feb	1332.87
1904-05	30-Dec	1936.87	30-Dec	1088.22	30-Dec	613.91	17-Mar	510.27	12-Mar	430.98
1905-06	18-Jan	2726.91	18-Jan	1958.68	16-Jan	1321.26	23-Mar	998.17	10-Mar	707.64
1906-07	19-Mar	5295.25	18-Mar	4256.87	18-Mar	2859.15	17-Mar	1772.92	1-Mar	1054.80
1907-08	3-Feb	461.56	2-Feb	421.92	20-Jan	328.19	20-Jan	292.80	14-Jan	254.00
1908-09	16-Jan	3879.41	14-Jan	3643.53	14-Jan	2533.51	8-Jan	1743.47	3-Jan	1166.37
1909-10	9-Dec	877.82	20-Mar	795.70	19-Mar	696.59	10-Mar	570.02	25-Feb	524.99
1910-11	31-Jan	2135.09	5-Apr	1463.98	2-Apr	1259.25	29-Mar	1036.40	17-Mar	749.26
1911-12	26-Jan	464.40	26-Jan	316.30	26-Jan	196.52	19-Jan	139.89	6-Mar	120.35
1912-13	18-Jan	305.82	18-Jan	229.37	14-Jan	190.57	13-Jan	137.62	13-Jan	105.91
1913-14	31-Dec	2495.00	31-Dec	2120.65	31-Dec	1322.11	31-Dec	816.94	31-Dec	740.77
1914-15	2-Feb	1246.22	1-Feb	895.38	29-Jan	575.68	28-Jan	459.02	28-Jan	400.68
1915-16	20-Mar	1220.17	20-Mar	1102.09	18-Mar	894.25	12-Mar	717.83	27-Feb	565.77
1916-17	25-Feb	2070.24	24-Feb	1411.88	22-Feb	855.45	21-Feb	531.22	31-Mar	489.88
1917-18	26-Mar	809.01	26-Mar	697.16	25-Mar	551.61	19-Mar	419.09	20-Mar	371.23
1918-19	11-Feb	1311.92	10-Feb	930.77	9-Feb	602.87	7-Feb	383.41	7-Feb	283.17
1919-20	16-Apr	605.41	15-Apr	518.20	14-Apr	374.63	8-Apr	291.10	27-Mar	218.04
1920-21	19-Nov	1466.53	18-Nov	997.32	17-Jan	636.85	17-Jan	493.56	4-Jan	392.19
1921-22	19-Feb	712.17	19-Feb	562.66	19-Feb	375.20	18-Feb	308.94	18-Feb	235.88
1922-23	13-Dec	456.75	12-Dec	378.88	10-Dec	273.82	6-Dec	195.67	6-Dec	178.11
1923-24	8-Feb	928.51	7-Feb	626.37	7-Feb	358.21	2-Feb	211.53	27-Jan	143.57
1924-25	6-Feb	1446.42	4-Feb	1236.88	4-Feb	839.88	3-Feb	526.69	3-Feb	367.27
1925-26	8-Apr	1320.98	7-Apr	1147.68	5-Apr	909.25	5-Apr	608.25	27-Mar	395.30
1926-27	21-Feb	2330.19	21-Feb	1751.40	17-Feb	1278.51	16-Feb	926.81	16-Feb	654.12
1927-28	26-Mar	3544.42	25-Mar	3139.77	23-Mar	2057.22	17-Mar	1119.36	2-Mar	647.04
1928-29	4-Feb	341.22	3-Feb	246.92	2-Feb	173.58	30-Jan	112.42	1-Feb	81.84
1929-30	15-Dec	2200.22	13-Dec	1729.31	11-Dec	1273.97	10-Dec	750.11	10-Dec	436.08
1930-31	19-Mar	275.52	18-Mar	238.71	18-Mar	193.69	11-Mar	165.09	1-Mar	117.51
1931-32	20-Mar	525.84	19-Mar	466.38	19-Mar	377.75	19-Mar	346.32	10-Mar	293.93
1932-33	29-Mar		28-Mar	203.03	12-Mar	154.33	16-Mar	133.37	2-Mar	116.95
1933-34	29-Mar	480.54	28-Mar	365.00	28-Mar	275.24	26-Mar	201.90	7-Feb	167.35
1934-35	8-Apr	1509.57	7-Apr	1164.39	4-Apr	887.17	4-Apr	763.99	3-Apr	656.10
1935-36	22-Feb	1616.04	21-Feb	1377.05	20-Feb	937.57	12-Feb	729.16	12-Feb	529.81
1936-37	12-Mar	442.31	12-Mar	402.95	11-Mar	321.11	11-Mar	295.91	12-Mar	257.68
1937-38	11-Dec	4501.81	10-Dec	3004.98	10-Dec	1704.96	10-Dec	929.93	2-Mar	634.86
1938-39	3-Dec	177.55	3-Dec	136.20	1-Dec	112.98	29-Nov	98.83	25-Nov	76.46
1939-40	30-Mar	3815.98	27-Feb	3055.67	27-Mar	1861.55	26-Mar	1181.66	14-Mar	734.26
1940-41	11-Feb	2075.91	10-Feb	1561.11	10-Feb	1038.38	10-Feb	729.16	9-Feb	664.88
1941-42	6-Feb	2523.60	5-Feb	1699.86	3-Feb	1218.47	25-Jan	1061.60	23-Jan	725.19
1942-43	23-Jan	1842.29	21-Jan	1782.83	21-Jan	1219.04	21-Jan	839.03	21-Jan	552.46
1943-44	4-Mar	530.66	4-Mar	334.14	29-Feb	235.88	25-Feb	173.02	9-Feb	130.82

1944-45	2-Feb	1348.73	2-Feb	1024.50	1-Feb	789.76	1-Feb	555.58	31-Jan	377.46
1945-46	29-Dec	1314.47	27-Dec	1192.42	24-Dec	961.64	22-Dec	766.54	21-Dec	499.23
1946-47	12-Feb	913.50	12-Feb	680.74	12-Feb	424.19	10-Feb	262.21	12-Feb	238.43
1947-48	17-Apr	943.52	16-Apr	776.16	16-Apr	650.15	15-Apr	558.13	5-Apr	452.79
1948-49	11-Mar	381.99	11-Mar	313.75	17-Mar	235.88	10-Mar	233.33	2-Mar	200.77
1949-50	6-Feb	1145.70	5-Feb	886.88	4-Feb	618.44	4-Feb	395.59	17-Jan	315.45
1950-51	21-Nov	1978.78	19-Nov	1490.32	17-Nov	1031.30	3-Dec	757.48	18-Nov	677.91
1951-52	2-Feb	1335.14	1-Feb	1055.09	1-Feb	714.72	31-Jan	504.32	24-Jan	421.64
1952-53	9-Jan	2799.12	9-Jan	1682.87	9-Jan	1225.84	8-Jan	910.67	7-Jan	595.79
1953-54	10-Mar	1363.17	9-Mar	1187.33	9-Mar	762.01	8-Mar	500.93	13-Mar	389.64
1954-55	15-Nov	236.45	6-Dec	192.55	3-Dec	158.29	2-Dec	123.74	15-Nov	99.96
1955-56	23-Dec	5140.36	22-Dec	4160.03	20-Dec	2774.48	19-Dec	1728.46	19-Dec	1200.63
1956-57	24-Feb	1787.08	24-Feb	1584.04	23-Feb	1042.63	23-Feb	741.62	23-Feb	512.82
1957-58	25-Feb	2169.92	24-Feb	1583.76	22-Feb	1060.18	13-Feb	902.74	3-Feb	708.77
1958-59	17-Feb	813.26	16-Feb	671.11	16-Feb	504.89	16-Feb	341.50	16-Feb	268.16
1959-60	8-Feb	2807.05	8-Feb	1544.40	7-Feb	891.98	2-Feb	515.65	8-Feb	363.59
1960-61	31-Jan	445.14	31-Jan	368.69	31-Jan	261.65	31-Jan	256.83	30-Jan	192.55
1961-62	10-Feb	1019.97	13-Feb	815.24	9-Feb	741.34	8-Feb	542.55	8-Feb	376.90
1962-63	1-Feb	3856.75	31-Jan	2833.38	31-Jan	1679.47	31-Jan	989.67	30-Jan	611.93
1963-64	21-Jan	579.65	20-Jan	429.85	19-Jan	263.06	19-Jan	182.93	18-Jan	137.90
1964-65	23-Dec	5055.97	22-Dec	4683.32	21-Dec	3197.82	21-Dec	1860.70	21-Dec	1247.92
1965-66	5-Jan	378.60	5-Jan	297.04	5-Jan	221.72	29-Dec	167.35	25-Dec	128.28
1966-67	30-Jan	1537.04	29-Jan	1456.62	28-Jan	997.32	21-Jan	766.54	21-Jan	516.50
1967-68	21-Feb	1136.92	21-Feb	1034.13	20-Feb	865.08	17-Feb	604.85	17-Feb	440.33
1968-69	21-Jan	3881.67	20-Jan	2883.79	20-Jan	1954.71	19-Jan	1256.42	13-Jan	848.37
1969-70	24-Jan	3332.33	22-Jan	2976.38	21-Jan	2315.75	14-Jan	1896.38	9-Jan	1203.18
1970-71	26-Mar	1823.04	26-Mar	1302.29	24-Mar	931.91	17-Mar	607.68	2-Mar	421.64
1971-72	29-Feb	566.05	3-Mar	486.20	29-Feb	458.73	28-Feb	405.50	23-Feb	342.35
1972-73	16-Jan	1368.84	16-Jan	1071.79	12-Jan	751.81	11-Jan	519.33	12-Jan	373.78
1973-74	30-Mar	3065.30	29-Mar	2258.27	28-Mar	1655.69	26-Mar	1099.54	12-Mar	763.42
1974-75	13-Feb	903.87	25-Mar	655.25	20-Mar	490.45	18-Mar	418.81	7-Mar	360.47
1975-76	29-Feb	342.07	29-Feb	293.93	28-Feb	207.56	27-Feb	153.19	28-Feb	127.14
1976-77	21-Feb	121.48	21-Feb	87.50	21-Feb	61.73	20-Feb	47.29	19-Feb	43.89
1977-78	16-Jan	1556.01	15-Jan	1365.44	14-Jan	1009.78	5-Jan	788.34	27-Dec	523.30
1978-79	14-Feb	662.90	13-Feb	428.43	13-Feb	311.49	13-Feb	247.21	14-Feb	227.67
1979-80	13-Jan	3896.96	12-Jan	3032.73	12-Jan	2052.41	11-Jan	1151.65	31-Dec	669.41
1980-81	14-Feb	534.06	27-Jan	413.99	14-Feb	327.63	13-Feb	250.60	27-Jan	207.56
1981-82	20-Dec	2800.54	19-Dec	2183.51	15-Feb	1360.06	13-Nov	1013.74	23-Nov	679.60
1982-83	13-Mar	2796.85	13-Mar	2020.41	11-Mar	1414.71	1-Mar	1249.91	26-Feb	965.04
1983-84	25-Dec	2115.55	25-Dec	1788.78	25-Dec	1278.79	24-Dec	859.42	9-Dec	669.98
1984-85	8-Feb	496.96	8-Feb	303.56	24-Nov	199.35	20-Nov	154.04	7-Nov	143.57
1985-86	17-Feb	6145.32	17-Feb	5295.53	15-Feb	3648.91	13-Feb	2202.48	14-Feb	1555.44
1986-87	13-Feb	877.26	12-Mar	600.60	12-Mar	438.34	5-Mar	343.48	5-Mar	246.07
1987-88	2-Dec	531.11	1-Dec	373.87	6-Dec	285.07	1-Dec	246.89	1-Dec	160.44
1988-89	10-Mar	2455.72	9-Mar	2235.19	8-Mar	1501.59	7-Mar	943.21	7-Mar	734.14
1989-90	13-Jan	416.99	13-Jan	319.56	27-May	250.26	17-Mar	192.89	2-Mar	187.43
1990-91	4-Mar	1408.14	3-Mar	944.68	2-Mar	549.80	1-Mar	346.60	3-Mar	253.15
1991-92	20-Feb	685.49	20-Feb	484.98	19-Feb	338.92	11-Feb	289.51	11-Feb	234.38
1992-93	18-Mar	1672.31	17-Mar	1461.77	13-Mar	1154.99	13-Mar	1003.27	13-Mar	756.51
1993-94	6-Mar	267.79	5-Mar	225.49	5-Mar	202.49	3-Mar	182.11	17-Feb	158.12
1994-95	10-Mar	3799.78	9-Mar	3221.98	9-Mar	2405.94	9-Mar	1692.16	9-Mar	1127.41

1995-96	5-Feb	1636.97	19-Feb	1396.25	18-Feb	1082.24	14-Feb	786.78	4-Feb	648.54
1996-97	1-Jan	8860.14	31-Dec	6923.04	29-Dec	4306.88	27-Dec	2409.34	29-Dec	1440.42
1997-98	3-Feb	1606.87	23-Mar	1243.53	2-Feb	957.39	2-Feb	715.11	11-Jan	650.86
1998-99	9-Feb	1654.13	8-Feb	1130.13	7-Feb	793.27	7-Feb	632.54	7-Feb	571.60
1999-00	14-Feb	1767.06	14-Feb	1191.18	11-Feb	800.69	14-Feb	628.29	11-Feb	535.25
2000-01	21-Feb	260.88	5-Mar	227.67	4-Mar	188.79	4-Mar	152.71	17-Feb	139.63
2001-02	3-Jan	623.62	2-Jan	540.12	2-Jan	461.62	29-Dec	359.85	17-Dec	251.77
2002-03	28-Dec	1209.81	27-Dec	876.49	27-Dec	622.06	15-Dec	438.51	27-Dec	397.74
2003-04	18-Feb	1869.96	17-Feb	1369.83	16-Feb	840.42	16-Feb	639.73	7-Feb	398.22
2004-05	23-Mar	676.35	22-Mar	594.91	22-Mar	478.41	20-Mar	415.10	13-Mar	333.66
2005-06	31-Dec	3899.48	31-Dec	2628.31	28-Dec	1928.75	22-Dec	1394.52	19-Dec	906.25
2006-07	10-Feb	907.16	9-Feb	751.47	9-Feb	499.51	9-Feb	313.72	7-Feb	239.48
2007-08	5-Jan	409.83	24-Jan	320.32	24-Jan	222.60	24-Jan	159.91	5-Jan	125.22

Appendix B – Important Oroville Dam Plans and Maps

This appendix contains a selection of drawings from (California Deparment of Water Resources, 1974).

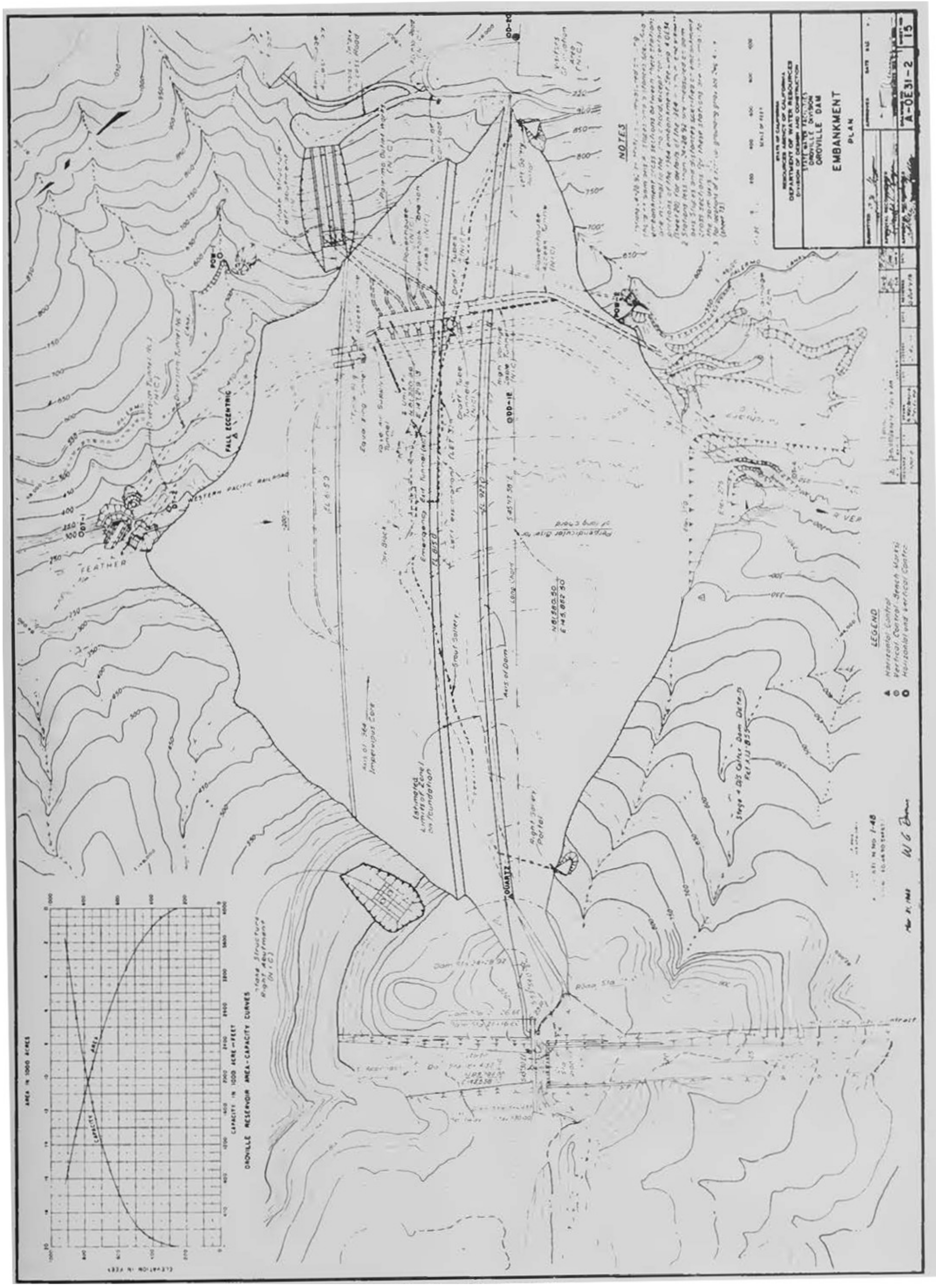

Figure 50. Oroville Dam Embankment Plan.

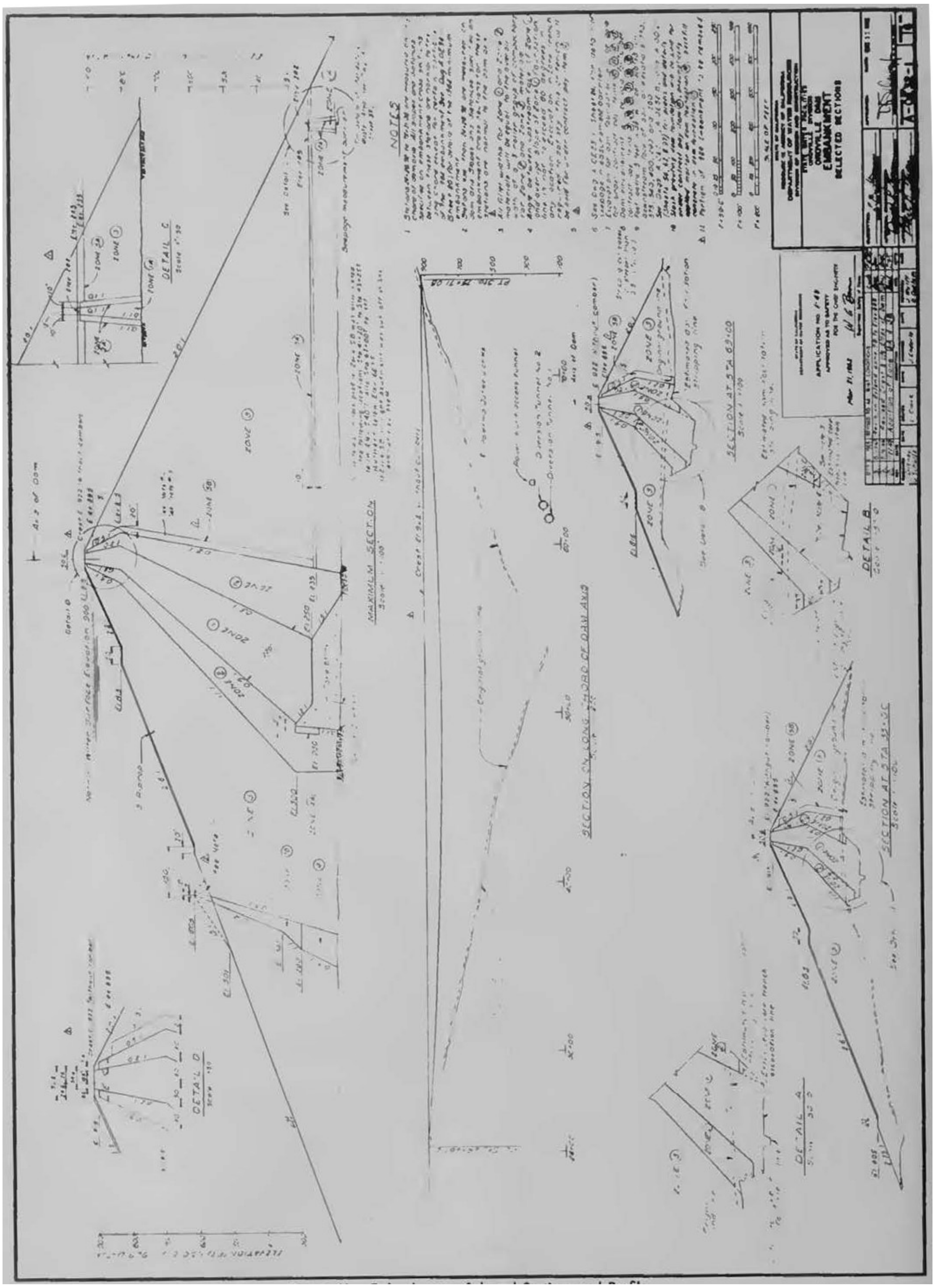

Figure 51. Embankment, Sections and Profile

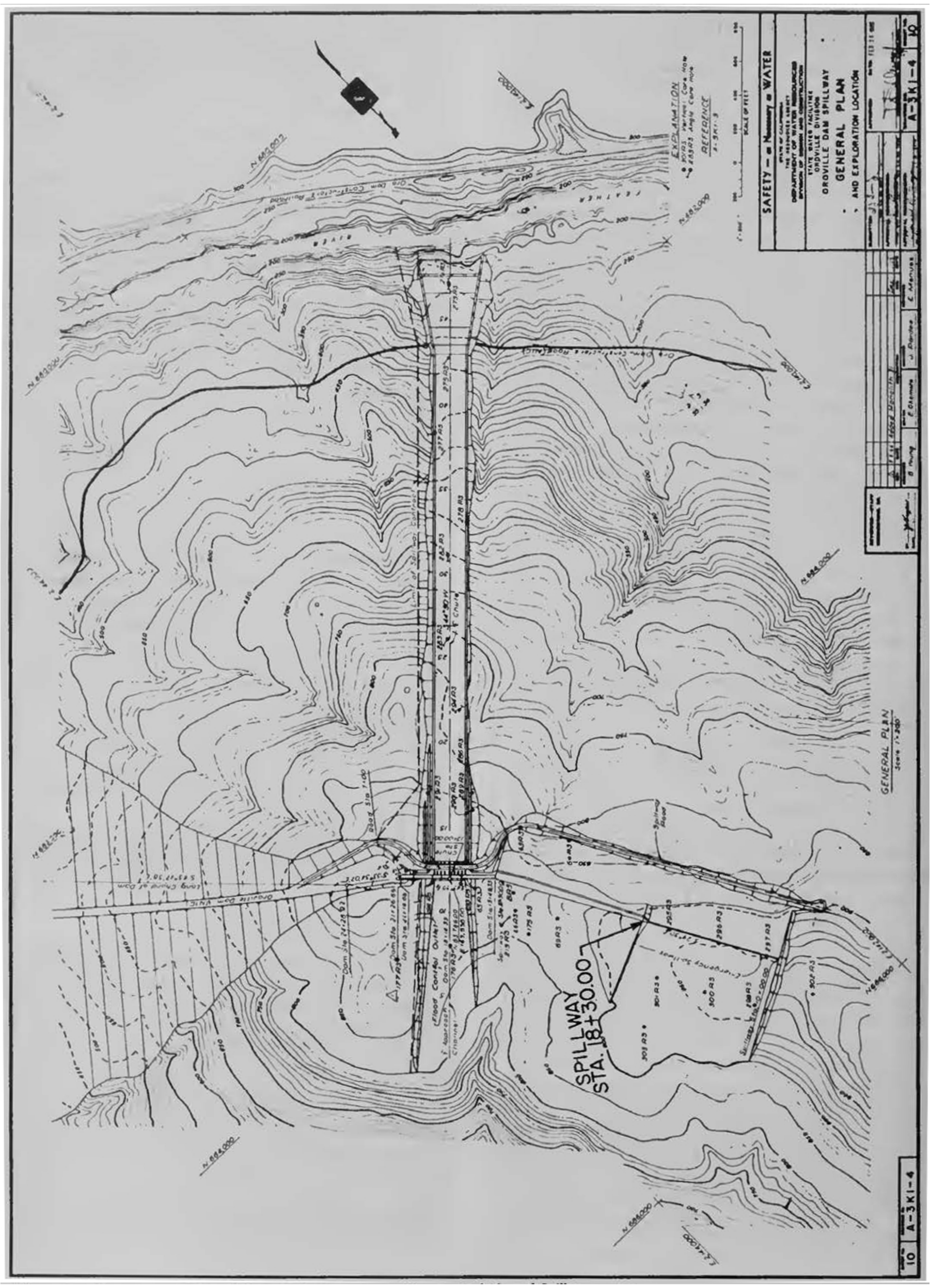

Figure 52. Spillway Plan.

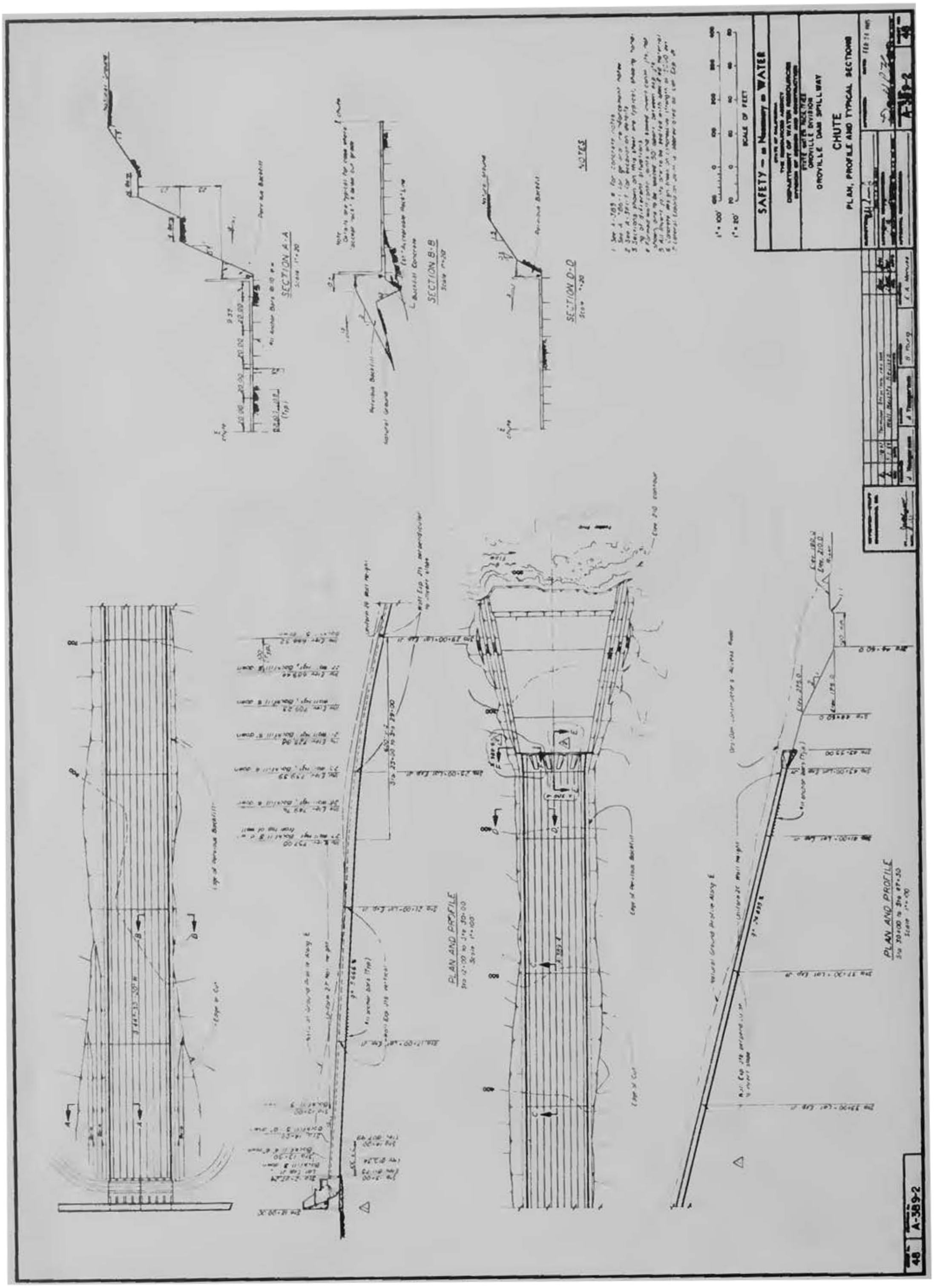

Figure 53. Main Spillway Chute Profile and Sections.

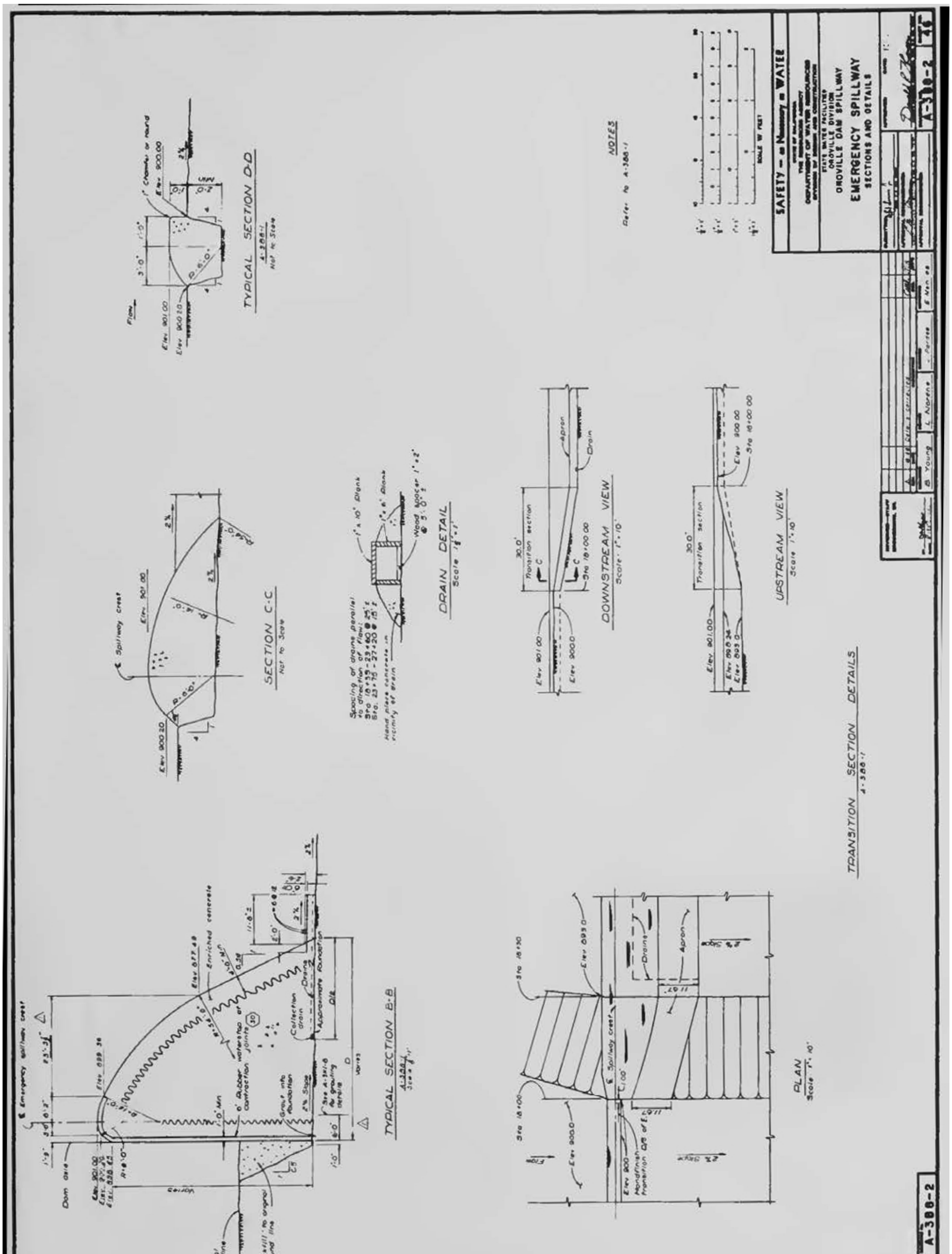

Figure 54. Emergency Spillway Sections and Details.

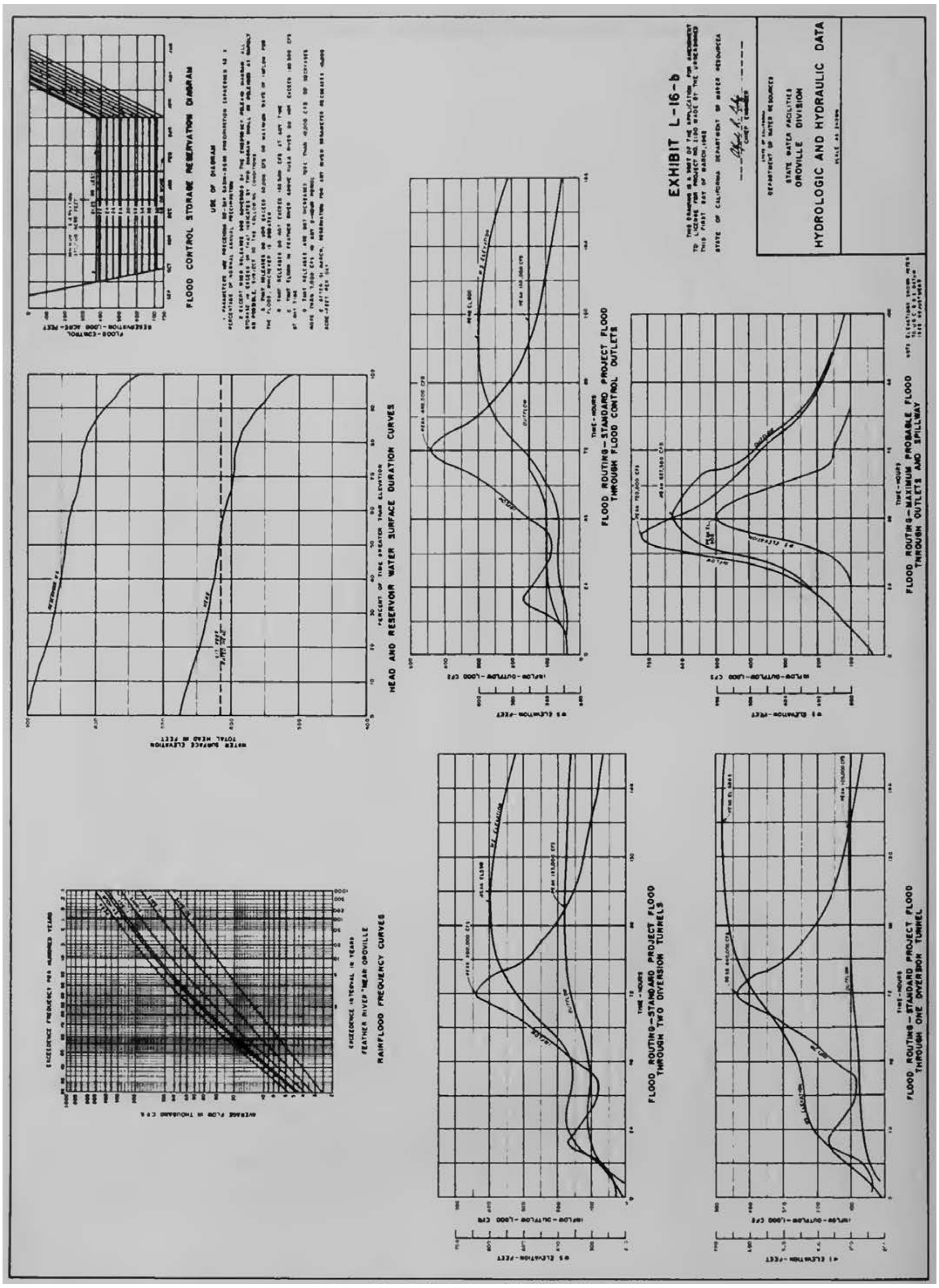

Figure 55. Hydrologic and Hydraulic Data.

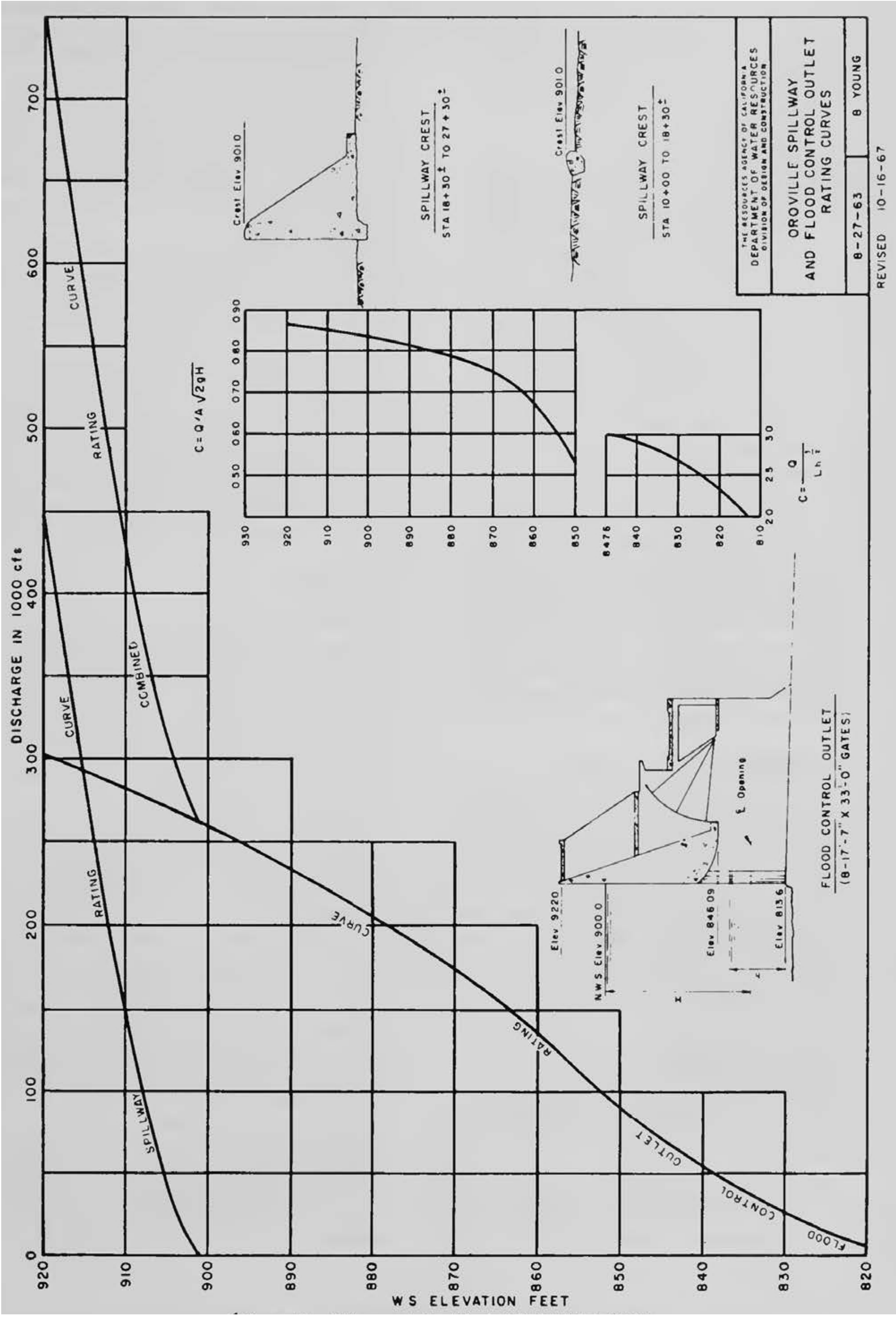

Figure 56. Spillway and Flood Control Rating Curves.

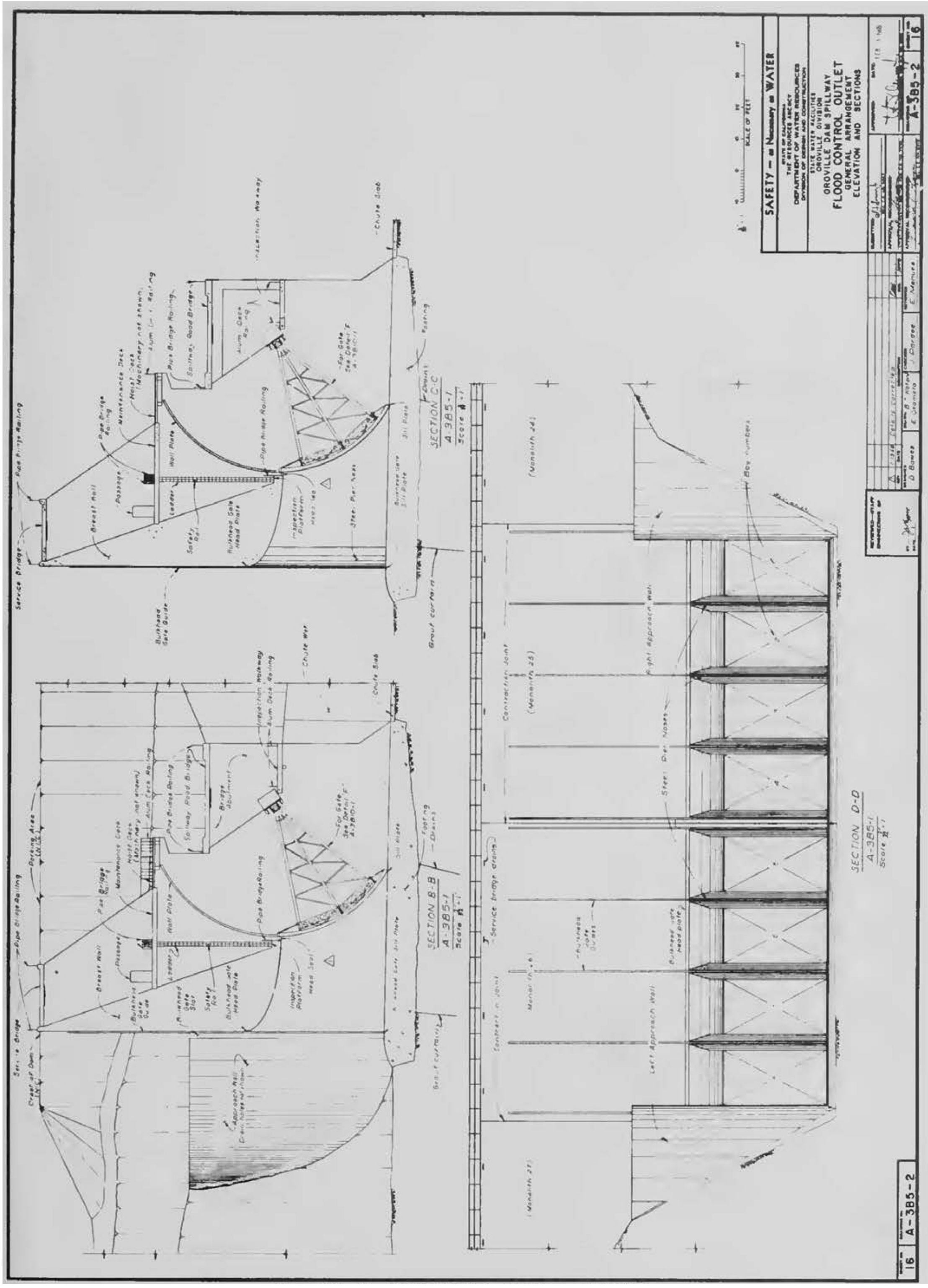

Figure 57. Flood Control Outlet Elevation and Sections.

Appendix C – Tabular Output of Spillway Chute 1-D Surface Water Profile Analysis

This section contains detailed data used for the construction of Oroville Dam's main spillway chute water surface profiles. For all analyses, spillway chute width is 187.67 feet, Manning's *n* coefficient is assumed to be 0.014, and standard gravity is assumed to be 32.2 ft/s^2.

Table C- 1. Tabular output of Oroville Dam main spillway 1-d surface water profile analysis, using the standard step method. Discharge is 20,000 cfs.

			Discharge 20,000 cfs			
Station	**Chute Invert Elevation**	**Top of Chute Wall Elevation**	**Flow Depth**	**Water Surface Elevation**	**Area**	**Velocity**
(ft.)	(ft.)	(ft.)	(ft.)	(ft.)	(ft^2)	(fps)
0	811.75	837.75	10.85	822.60	1937.20	10.32
50	809.87	835.87	4.56	814.43	814.04	24.57
100	807.99	833.99	4.29	812.28	765.57	26.12
150	805.16	831.16	3.97	809.13	707.81	28.26
200	802.32	828.32	3.75	806.07	669.64	29.87
250	799.48	825.48	3.60	803.08	642.98	31.10
300	796.65	822.65	3.50	800.15	623.83	32.06
350	793.82	819.82	3.42	797.24	609.61	32.81
400	790.98	816.98	3.36	794.34	598.70	33.41
450	788.15	814.15	3.31	791.46	589.73	33.91
500	785.31	811.31	3.27	788.58	582.86	34.31
550	782.47	808.47	3.24	785.71	577.67	34.62
600	779.64	805.64	3.22	782.86	573.88	34.85
650	776.80	802.80	3.20	780.00	570.85	35.04
700	773.97	799.97	3.19	777.16	568.67	35.17
750	771.14	797.14	3.18	774.32	567.00	35.27
800	768.30	794.30	3.17	771.47	565.58	35.36
850	765.46	791.46	3.16	768.62	564.48	35.43
900	762.63	788.63	3.16	765.79	563.80	35.47
950	759.82	785.82	3.16	762.98	563.53	35.49
1000	757.00	783.00	3.16	760.16	563.14	35.51
1050	753.38	778.38	3.10	756.48	552.17	36.22
1100	749.76	773.76	3.06	752.82	544.09	36.76
1150	744.58	767.58	2.93	747.51	520.69	38.41
1200	739.39	761.39	2.85	742.24	504.77	39.62

1250	732.64	753.64	2.71	735.35	480.59	41.62
1300	725.88	745.88	2.64	728.52	464.77	43.03
1350	717.55	736.55	2.52	720.07	444.06	45.04
1400	709.23	727.23	2.46	711.69	430.87	46.42
1450	699.33	716.33	2.36	701.69	414.19	48.29
1500	689.44	705.44	2.32	691.76	403.69	49.54
1550	677.98	693.98	2.24	680.22	390.36	51.23
1600	666.52	682.52	2.20	668.72	382.07	52.35
1650	654.28	670.28	2.16	656.44	374.03	53.47
1700	642.03	658.03	2.13	644.16	368.96	54.21
1750	629.79	645.79	2.11	631.90	365.79	54.68
1800	617.54	633.54	2.10	619.64	363.73	54.99
1850	605.29	621.29	2.09	607.38	362.42	55.18
1900	593.04	609.04	2.08	595.12	361.57	55.31
1950	580.79	596.79	2.08	582.87	361.03	55.40
2000	568.54	584.54	2.08	570.62	360.68	55.45
2050	556.29	572.29	2.08	558.37	360.45	55.49
2100	544.04	560.04	2.08	546.12	360.31	55.51
2150	531.79	547.79	2.08	533.87	360.21	55.52
2200	519.54	535.54	2.08	521.62	360.15	55.53
2250	507.29	523.29	2.08	509.37	360.11	55.54
2300	495.04	511.04	2.07	497.11	360.09	55.54
2350	482.79	498.79	2.07	484.86	360.07	55.54
2400	470.55	486.55	2.08	472.63	360.09	55.54
2450	458.30	474.30	2.07	460.37	360.08	55.54
2500	446.05	462.05	2.07	448.12	360.06	55.55
2550	433.80	449.80	2.07	435.87	360.06	55.55
2600	421.55	437.55	2.07	423.62	360.05	55.55
2650	409.30	425.30	2.07	411.37	360.05	55.55
2700	397.05	413.05	2.07	399.12	360.05	55.55
2750	384.80	400.80	2.07	386.87	360.05	55.55
2800	372.55	388.55	2.07	374.62	360.04	55.55
2850	360.30	376.30	2.07	362.37	360.04	55.55
2900	348.05	364.05	2.07	350.12	360.04	55.55
2950	335.80	351.80	2.07	337.87	360.04	55.55
3000	323.55	339.55	2.07	325.62	360.04	55.55

Table C- 2. Tabular output of Oroville Dam main spillway 1-d surface water profile analysis, using the standard step method. Discharge is 50,000 cfs.

			Discharge 50,000 cfs			
Station	**Chute Invert Elevation**	**Top of Chute Wall Elevation**	**Flow Depth**	**Water Surface Elevation**	**Area**	**Velocity**
(ft.)	(ft.)	(ft.)	(ft.)	(ft.)	(ft^2)	(fps)
0	811.75	837.75	19.99	831.74	3569.09	14.01
50	809.87	835.87	8.78	818.65	1568.32	31.88
100	807.99	833.99	8.37	816.36	1493.03	33.49
150	805.16	831.16	7.87	813.03	1403.60	35.62
200	802.32	828.32	7.49	809.81	1335.98	37.43
250	799.48	825.48	7.19	806.67	1282.97	38.97
300	796.65	822.65	6.95	803.60	1240.48	40.31
350	793.82	819.82	6.76	800.58	1205.53	41.48
400	790.98	816.98	6.59	797.57	1176.15	42.51
450	788.15	814.15	6.44	794.59	1149.00	43.52
500	785.31	811.31	6.31	791.62	1126.12	44.40
550	782.47	808.47	6.20	788.67	1106.80	45.18
600	779.64	805.64	6.11	785.75	1090.56	45.85
650	776.80	802.80	6.03	782.83	1076.51	46.45
700	773.97	799.97	5.97	779.94	1064.59	46.97
750	771.14	797.14	5.91	777.05	1054.31	47.42
800	768.30	794.30	5.86	774.16	1045.27	47.83
850	765.46	791.46	5.82	771.28	1037.41	48.20
900	762.63	788.63	5.78	768.41	1030.72	48.51
950	759.82	785.82	5.75	765.57	1025.18	48.77
1000	757.00	783.00	5.72	762.72	1020.11	49.01
1050	753.38	778.38	5.64	759.02	1004.93	49.75
1100	749.76	773.76	5.58	755.34	991.87	50.41
1150	744.58	767.58	5.42	750.00	962.38	51.95
1200	739.39	761.39	5.30	744.69	938.27	53.29
1250	732.64	753.64	5.10	737.74	903.73	55.33
1300	725.88	745.88	4.97	730.85	876.34	57.06
1350	717.55	736.55	4.78	722.33	842.67	59.33
1400	709.23	727.23	4.66	713.89	816.52	61.24
1450	699.33	716.33	4.49	703.82	786.43	63.58
1500	689.44	705.44	4.38	693.82	763.20	65.51
1550	677.98	693.98	4.23	682.21	737.39	67.81
1600	666.52	682.52	4.13	670.65	717.55	69.68
1650	654.28	670.28	4.03	658.31	698.94	71.54
1700	642.03	658.03	3.94	645.97	684.48	73.05
1750	629.79	645.79	3.88	633.67	673.19	74.27

1800	617.54	633.54	3.83	621.37	664.25	75.27
1850	605.29	621.29	3.79	609.08	657.16	76.09
1900	593.04	609.04	3.75	596.79	651.50	76.75
1950	580.79	596.79	3.73	584.52	646.97	77.28
2000	568.54	584.54	3.71	572.25	643.34	77.72
2050	556.29	572.29	3.69	559.98	640.42	78.07
2100	544.04	560.04	3.68	547.72	638.07	78.36
2150	531.79	547.79	3.67	535.46	636.17	78.60
2200	519.54	535.54	3.66	523.20	634.64	78.79
2250	507.29	523.29	3.65	510.94	633.40	78.94
2300	495.04	511.04	3.64	498.68	632.39	79.06
2350	482.79	498.79	3.64	486.43	631.58	79.17
2400	470.55	486.55	3.64	474.19	630.95	79.25
2450	458.30	474.30	3.63	461.93	630.41	79.31
2500	446.05	462.05	3.63	449.68	629.98	79.37
2550	433.80	449.80	3.63	437.43	629.62	79.41
2600	421.55	437.55	3.63	425.18	629.34	79.45
2650	409.30	425.30	3.63	412.93	629.10	79.48
2700	397.05	413.05	3.62	400.67	628.91	79.50
2750	384.80	400.80	3.62	388.42	628.76	79.52
2800	372.55	388.55	3.62	376.17	628.64	79.54
2850	360.30	376.30	3.62	363.92	628.53	79.55
2900	348.05	364.05	3.62	351.67	628.45	79.56
2950	335.80	351.80	3.62	339.42	628.39	79.57
3000	323.55	339.55	3.62	327.17	628.39	79.57

Table C- 3. Tabular output of Oroville Dam main spillway 1-d surface water profile analysis, using the standard step method. Discharge is 100,000 cfs.

			Discharge 100,000 cfs			
Station	**Chute Invert Elevation**	**Top of Chute Wall Elevation**	**Flow Depth**	**Water Surface Elevation**	**Area**	**Velocity**
(ft.)	(ft.)	(ft.)	(ft.)	(ft.)	(ft^2)	(fps)
0	811.75	837.75	31.73	843.48	5665.20	17.65
50	809.87	835.87	14.29	824.16	2551.99	39.19
100	807.99	833.99	13.78	821.77	2458.18	40.68
150	805.16	831.16	13.14	818.30	2343.80	42.67
200	802.32	828.32	12.62	814.94	2250.95	44.43
250	799.48	825.48	12.19	811.67	2173.70	46.00
300	796.65	822.65	11.82	808.47	2108.48	47.43
350	793.82	819.82	11.51	805.33	2052.30	48.73
400	790.98	816.98	11.23	802.21	2003.08	49.92
450	788.15	814.15	10.96	799.11	1955.38	51.14
500	785.31	811.31	10.73	796.04	1913.47	52.26
550	782.47	808.47	10.52	792.99	1876.54	53.29
600	779.64	805.64	10.34	789.98	1844.04	54.23
650	776.80	802.80	10.17	786.97	1814.86	55.10
700	773.97	799.97	10.03	784.00	1788.93	55.90
750	771.14	797.14	9.90	781.04	1765.60	56.64
800	768.30	794.30	9.78	778.08	1744.39	57.33
850	765.46	791.46	9.67	775.13	1725.19	57.96
900	762.63	788.63	9.57	772.20	1707.93	58.55
950	759.82	785.82	9.49	769.31	1692.56	59.08
1000	757.00	783.00	9.42	766.42	1678.22	59.59
1050	753.38	778.38	9.28	762.66	1652.87	60.50
1100	749.76	773.76	9.17	758.93	1629.84	61.36
1150	744.58	767.58	8.94	753.52	1588.12	62.97
1200	739.39	761.39	8.76	748.15	1551.57	64.45
1250	732.64	753.64	8.48	741.12	1502.32	66.56
1300	725.88	745.88	8.29	734.17	1460.19	68.48
1350	717.55	736.55	8.00	725.55	1410.00	70.92
1400	709.23	727.23	7.80	717.03	1367.79	73.11
1450	699.33	716.33	7.53	706.86	1320.55	75.73
1500	689.44	705.44	7.36	696.80	1281.04	78.06
1550	677.98	693.98	7.11	685.09	1238.48	80.74
1600	666.52	682.52	6.93	673.45	1203.01	83.12
1650	654.28	670.28	6.74	661.02	1169.40	85.51
1700	642.03	658.03	6.58	648.61	1141.17	87.63
1750	629.79	645.79	6.44	636.23	1117.33	89.50

1800	617.54	633.54	6.32	623.86	1097.00	91.16
1850	605.29	621.29	6.22	611.51	1079.57	92.63
1900	593.04	609.04	6.13	599.17	1064.58	93.93
1950	580.79	596.79	6.06	586.85	1051.62	95.09
2000	568.54	584.54	6.00	574.54	1040.39	96.12
2050	556.29	572.29	5.94	562.23	1030.62	97.03
2100	544.04	560.04	5.89	549.93	1022.11	97.84
2150	531.79	547.79	5.85	537.64	1014.67	98.55
2200	519.54	535.54	5.81	525.35	1008.16	99.19
2250	507.29	523.29	5.78	513.07	1002.45	99.76
2300	495.04	511.04	5.75	500.79	997.44	100.26
2350	482.79	498.79	5.72	488.51	993.04	100.70
2400	470.55	486.55	5.70	476.25	989.20	101.09
2450	458.30	474.30	5.68	463.98	985.78	101.44
2500	446.05	462.05	5.66	451.71	982.77	101.75
2550	433.80	449.80	5.65	439.45	980.12	102.03
2600	421.55	437.55	5.63	427.18	977.78	102.27
2650	409.30	425.30	5.62	414.92	975.71	102.49
2700	397.05	413.05	5.61	402.66	973.88	102.68
2750	384.80	400.80	5.60	390.40	972.27	102.85
2800	372.55	388.55	5.59	378.14	970.84	103.00
2850	360.30	376.30	5.59	365.89	969.58	103.14
2900	348.05	364.05	5.58	353.63	968.46	103.26
2950	335.80	351.80	5.58	341.38	967.47	103.36
3000	323.55	339.55	5.58	329.13	967.47	103.36

Table C- 4. Tabular output of Oroville Dam main spillway 1-d surface water profile analysis, using the standard step method. Discharge is 277,000 cfs.

			Discharge 277,000 cfs			
Station	**Chute Invert Elevation**	**Top of Chute Wall Elevation**	**Flow Depth**	**Water Surface Elevation**	**Area**	**Velocity**
(ft.)	(ft.)	(ft.)	(ft.)	(ft.)	(ft^2)	(fps)
0	811.75	837.75	62.57	874.32	11171.49	24.80
50	809.87	835.87	28.88	838.75	5156.27	53.72
100	807.99	833.99	28.26	836.25	5041.31	54.95
150	805.16	831.16	27.44	832.60	4895.02	56.59
200	802.32	828.32	26.72	829.04	4766.34	58.12
250	799.48	825.48	26.08	825.56	4652.15	59.54
300	796.65	822.65	25.51	822.16	4550.20	60.88
350	793.82	819.82	24.99	818.81	4458.00	62.14
400	790.98	816.98	24.52	815.50	4373.63	63.33
450	788.15	814.15	24.04	812.19	4287.64	64.60
500	785.31	811.31	23.60	808.91	4209.02	65.81
550	782.47	808.47	23.19	805.66	4137.07	66.96
600	779.64	805.64	22.82	802.46	4071.23	68.04
650	776.80	802.80	22.48	799.28	4010.12	69.08
700	773.97	799.97	22.16	796.13	3953.81	70.06
750	771.14	797.14	21.87	793.01	3901.41	71.00
800	768.30	794.30	21.60	789.90	3852.28	71.91
850	765.46	791.46	21.34	786.80	3806.37	72.77
900	762.63	788.63	21.10	783.73	3763.62	73.60
950	759.82	785.82	20.88	780.70	3723.96	74.38
1000	757.00	783.00	20.68	777.68	3685.92	75.15
1050	753.38	778.38	20.38	773.76	3632.52	76.26
1100	749.76	773.76	20.15	769.91	3581.78	77.34
1150	744.58	767.58	19.72	764.30	3504.32	79.05
1200	739.39	761.39	19.39	758.78	3432.57	80.70
1250	732.64	753.64	18.87	751.51	3341.72	82.89
1300	725.88	745.88	18.49	744.37	3259.09	84.99
1350	717.55	736.55	17.95	735.50	3163.65	87.56
1400	709.23	727.23	17.56	726.79	3078.02	89.99
1450	699.33	716.33	17.03	716.36	2984.18	92.82
1500	689.44	705.44	16.65	706.09	2900.43	95.50
1550	677.98	693.98	16.15	694.13	2811.76	98.51
1600	666.52	682.52	15.75	682.27	2733.18	101.35
1650	654.28	670.28	15.31	669.59	2657.53	104.23
1700	642.03	658.03	14.93	656.96	2590.31	106.94
1750	629.79	645.79	14.58	644.37	2530.30	109.47

1800	617.54	633.54	14.27	631.81	2476.31	111.86
1850	605.29	621.29	13.99	619.28	2427.56	114.11
1900	593.04	609.04	13.73	606.77	2383.35	116.22
1950	580.79	596.79	13.50	594.29	2343.12	118.22
2000	568.54	584.54	13.29	581.83	2306.37	120.10
2050	556.29	572.29	13.10	569.39	2272.72	121.88
2100	544.04	560.04	12.92	556.96	2241.81	123.56
2150	531.79	547.79	12.75	544.54	2213.36	125.15
2200	519.54	535.54	12.60	532.14	2187.11	126.65
2250	507.29	523.29	12.46	519.75	2162.86	128.07
2300	495.04	511.04	12.33	507.37	2140.40	129.42
2350	482.79	498.79	12.21	495.00	2119.57	130.69
2400	470.55	486.55	12.10	482.65	2100.26	131.89
2450	458.30	474.30	12.00	470.30	2082.27	133.03
2500	446.05	462.05	11.90	457.95	2065.52	134.11
2550	433.80	449.80	11.81	445.61	2049.90	135.13
2600	421.55	437.55	11.73	433.28	2035.33	136.10
2650	409.30	425.30	11.65	420.95	2021.71	137.01
2700	397.05	413.05	11.58	408.63	2008.99	137.88
2750	384.80	400.80	11.51	396.31	1997.08	138.70
2800	372.55	388.55	11.44	383.99	1985.93	139.48
2850	360.30	376.30	11.38	371.68	1975.48	140.22
2900	348.05	364.05	11.33	359.38	1965.68	140.92
2950	335.80	351.80	11.27	347.07	1956.48	141.58
3000	323.55	339.55	11.25	334.80	1952.30	141.88

Appendix D – Annual Maxima of Daily Rainfall Time Series

Table D- 1. Annual daily maxima of precipitation (mm), Brush Creek (BRS) station.

Year	Annual Maximum (mm)
1986	217.4
1987	121.9
1988	
1989	89.4
1990	93.2
1991	122.4
1992	89.4
1993	170.2
1994	76.5
1995	141.5
1996	179.1
1997	285.2
1998	115.1
1999	94.5
2000	130.3
2001	108.0
2002	211.8
2003	122.4
2004	113.8
2005	162.8
2006	124.5
2007	109.5
2008	100.8
2009	86.4
2010	206.5
2011	87.9
2012	163.8
2013	56.9
2014	147.1
2015	92.2
2016	218.4
2017	135.6

Table D- 2. Annual daily maxima of precipitation (mm), station USC00044812.

Year	Annual Maximum (mm)	Year	Annual Maximum (mm)
1913	127	**1954**	109.2
1914	44.5	**1955**	189.2
1915	129.5	**1956**	125.5
1916	125.5	**1957**	119.9
1917	96.5	**1958**	89.7
1918	68.6	**1959**	77.5
1919	96.5	**1960**	66.8
1920	107.2	**1961**	79.5
1921	76.2	**1962**	239
1922	116.1	**1963**	133.9
1923	61.5	**1964**	156.2
1924	101.6	**1965**	88.9
1925	97.3	**1966**	87.4
1926	132.3	**1967**	134.9
1927	80.3		
1928	76.2		
1929	123.2		
1930	57.7		
1931	97.8		
1932	57.2		
1933	77.7		
1934	63.5		
1935	114.3		
1936	98		
1937	191.5		
1938	85.1		
1939	81.8		
1940	194.6		
1941	116.8		
1942	114.3		
1943	162.1		
1944	88.9		
1945	92.7		
1946	61.5		
1947	113.8		
1948	64.3		
1949	70.4		
1950	144.8		
1951	95.3		
1952	104.1		
1953	115.8		

Table D- 3. Annual daily maxima of precipitation (mm), station USC00041159

Year	Annual Daily Maximum (mm)	Year	Annual Daily Maximum (mm)
1959	98.8	**1989**	130.6
1960	168.4	**1990**	91.7
1961	96.8	**1991**	106.2
1962	271.8	**1992**	98.3
1963	125.5	**1993**	103.6
1964	254.5	**1994**	119.9
1965	117.9	**1995**	210.8
1966	109.2	**1996**	145.5
1967	142.5	**1997**	61.5
1968	97.3	**1998**	174
1969	121.9	**1999**	100.3
1970	115.1	**2001**	105.4
1971	100.3	**2002**	119.4
1972	50.8	**2003**	101.6
1973	160.5	**2004**	102.4
1974	102.4	**2005**	153.4
1975	82	**2006**	94.5
1976	66.8	**2007**	57.4
1977	70.4	**2008**	106.9
1978	136.1	**2009**	86.6
1979	135.9	**2010**	85.3
1980	157.5	**2011**	95.5
1981	154.9	**2012**	119.9
1982	218.4	**2013**	37.8
1983	117.9	**2014**	112.5
1984	88.9	**2015**	88.1
1985	86.6	**2016**	66.8
1986	123.2		
1987	88.4		
1988	132.8		

Table D- 4. Annual daily maxima of precipitation (mm), Quincy (QCY) station.

Year	Annual Daily Maximum (mm)
1989	121.9
1990	80.3
1991	95.5
1992	38.6
1993	201.7
1994	43.7
1995	
1996	239.8
1997	86.4
1998	96.5
1999	46.2
2000	65.5
2001	63.5
2002	110.5
2003	74.7
2004	93.7
2005	77.0
2006	101.3
2007	49.3
2008	104.1
2009	90.2
2010	54.1
2011	61.5
2012	106.7
2013	31.5
2014	117.1
2015	76.5
2016	110.7
2017	123.7