# WHY WASN'T CASTLEWOOD WORTH A DAM?

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## INTRODUCTION

Castlewood Dam was a masonry rock-fill dam constructed in 1890 across Cherry Creek approximately 30 miles southeast of Denver, Colorado. The dam contained an uncontrolled overflow spillway passing over the dam at about the middle of the crest length. The dam failed at approximately 12:15 am on August 3, 1933 after a heavy rainfall. The flood resulting from the dam break flowed through downtown Denver. The dam site and remains are now located within the boundaries of Castlewood Canyon State Park in Douglas County, as shown in Figure 1. After the dam was built, its safety was questioned by citizens downstream due to a number of problems with leakage, settlement, and cracking during its first 12 years of operation. Despite these initial problems, Castlewood Dam survived for over 42 years with only minor leakage issues during the last 30 years of its life. Following the failure, a few questions as to its exact cause remained. In this article we explore the cause of failure based on newly found evidence and modern evaluation techniques.

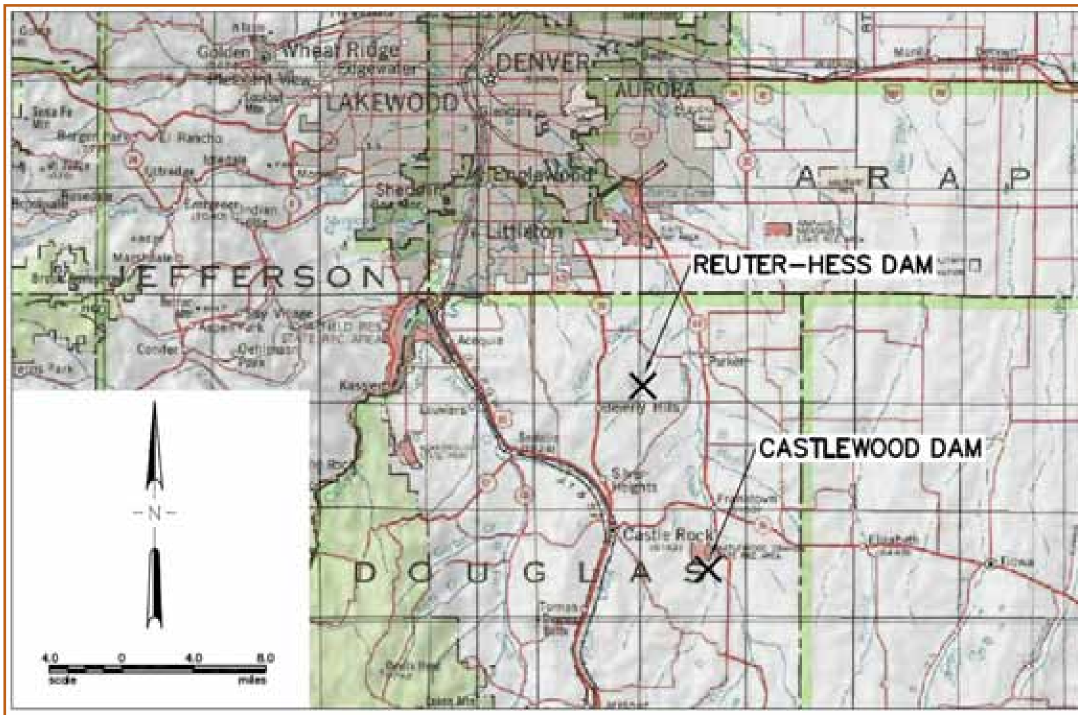


**Figure 1. Location of Castlewood Dam**

## Geologic Setting

Castlewood Dam lies in an area where the bedrock is comprised primarily of the Upper Dawson Formation, consisting of layers of arkosic sandstone and pebble conglomerates interbedded with finer grained mudflow channel deposits and fine grained sandy claystone. The bedrock is overlain by relatively thin layers of Quaternary alluvium near the center of drainages and thin layers of colluvium overlying steep slopes near the sides of the drainages. The Castle Rock Conglomerate overlies the Upper Dawson Formation in the vicinity of Castlewood Dam. This conglomerate forms cliffs above the dam on both sides of Cherry Creek. Large blocks of the conglomerate were used to construct the dam.

## Design and Construction

The idea of constructing the Castlewood Dam for irrigation purposes was first raised by the Colorado General Assembly in early 1889. Shortly after this, the Denver Land and Water Storage Company was formed and plans to build the dam were initiated (Friedman, 1987). It was anticipated that a dam placed in this location would collect rainfall runoff over a 175 square mile watershed and provide irrigation water for at least 16,000 acres. During the summer of 1889, plans for the dam were prepared by A.M. Welles, an engineer hired by the Denver Land and Water Storage Company. In general, the dam was proposed as a rock-fill masonry structure consisting of rubble fill

**Figure 2. Construction photo (1890) looking toward the right abutment (Photo Courtesy of the Denver Public Library, Western History Collection)**





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hand laid between mortared masonry walls. Details of the design intent before construction began are limited as no plans were released until after construction was completed.

Construction of the Castlewood Dam commenced in December 1889 and continued over the next 11 months until completion in November 1890. With the exception of a few low quality photographs, very little documentation of the construction exists. Figure 2 is a photograph taken during construction showing the stiff-legged derricks used to move the large stone into position. Final design drawings, as shown in Figure 3, were not released until 1898 when the design engineer published an article regarding the dam in the Engineering Record.

As constructed, the Castlewood Dam was reported to be approximately 600 ft long, 70 ft high from the lowest point of the reservoir floor, 92 ft high above the lowest point of the foundation on the upstream masonry wall, 50 ft wide at the base near the center, and 8 ft wide at the top. The main center spillway was 100 ft long and 4 ft deep. A 40-ft wide masonry lined bypass spillway capable of passing 4,000 cfs was also constructed near the left abutment. The upstream masonry wall was constructed at a 1:10 (H:V) slope from conglomerate blocks set in mortar. The wall had a thickness of 11 ft at the base and 4 ft at the crest along the center spillway, and a consistent thickness of 4 ft from base to the crest along the rest of the dam. The upstream masonry wall was reported to be founded 6 to 22

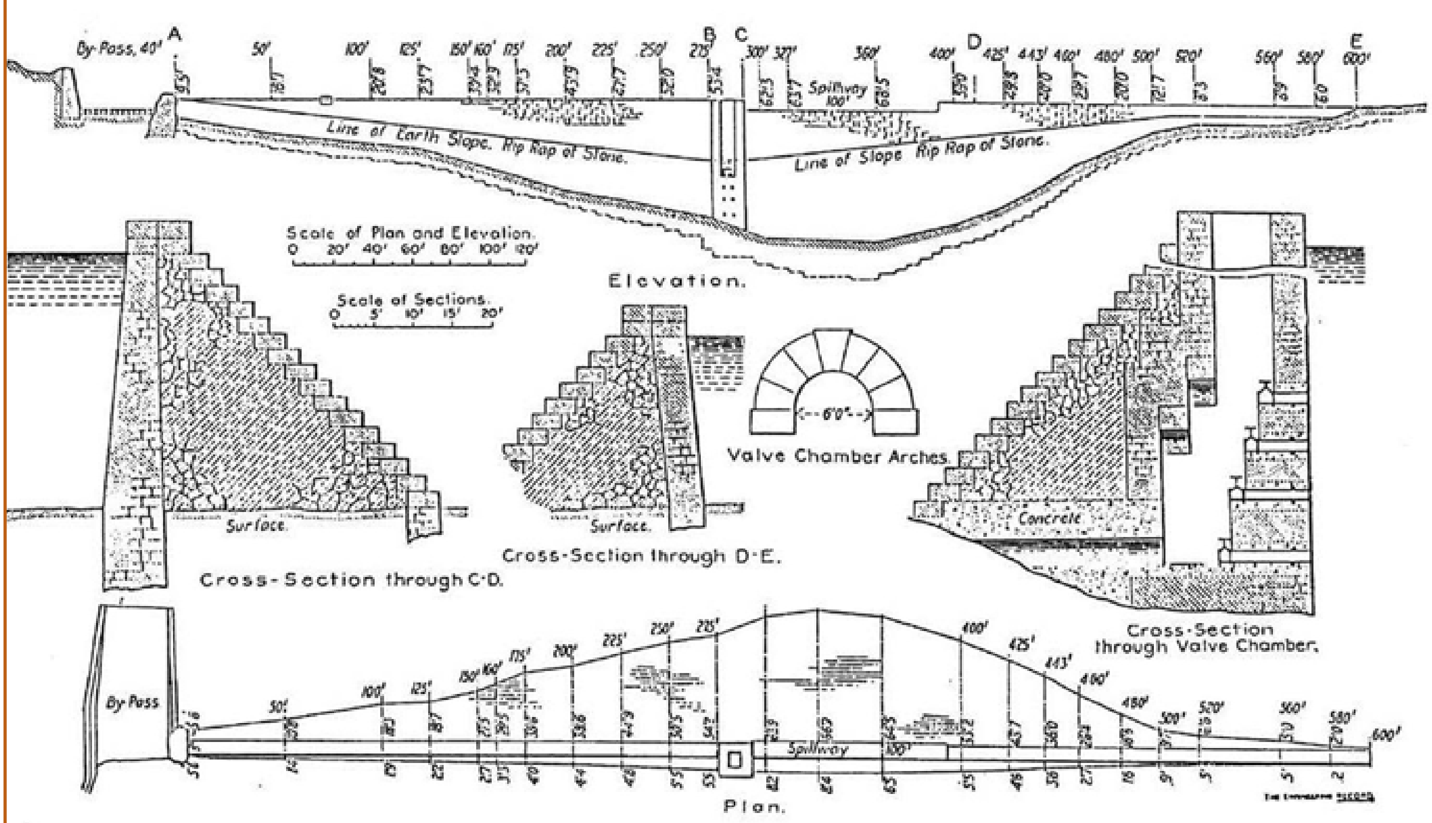


**Figure 3. Plans for Castlewood Canyon Dam (Welles, 1898, Reprinted Courtesy of Engineering News Record).**

ft below ground surface on “hard sandy clay.” The “hard sandy clay” referred to in the report is likely a mudstone unit within the Dawson Formation. The downstream masonry wall was constructed at a 1:1 (H:V) slope from 2 ft thick conglomerate blocks set in mortar. Construction of the downstream masonry wall was completed by layering the 2 ft thick conglomerate blocks on one another in rows. Each row stepped in approximately 2 ft resulting in a stepped surface with 2 ft by 2 ft steps. The foundation of the downstream masonry wall was reported to be founded 10 ft below the ground surface. No information on the foundation of the downstream masonry wall was reported. The rubble fill between the masonry walls was placed directly on the ground surface and consisted of loose dumped rock of various sizes. An outlet control structure consisting of eight 12 inch pipes with a total combined capacity of approximately 250 cfs was constructed near the center of the dam. These pipes fed into a square control tower, which connected to a 36 inch outlet pipe discharging near the downstream toe of the dam (Bartlett, 2003; U.S. Geological Survey, 1899). During initial construction, a small soil berm was placed along the upstream masonry wall.

## Performance History

A partial failure of the dam occurred in May of 1897 due to waves washing against the upper upstream masonry wall. The toe of the wall was undermined on the east side (right abutment), causing settlement and several horizontal cracks 2 to 4 inches wide. Substantial flows through the dam were observed and the reservoir was ordered to be emptied (Hardesty, 1899). In August of 1897, a large rainstorm filled the reservoir (still under order to be empty). Water poured through the previously formed cracks, ran down the right abutment groin, and undermined the downstream toe of the dam so that it settled downward and outward.

During the summer of 1898, repairs were made to the dam. These repairs consisted of removal and replacement of heavily cracked portions of the masonry, placement of a clay puddle wall against the upstream masonry wall, raising the upstream embankment, placement of riprap along the right abutment groin and upstream embankment, and placement of a 25 ft wide, 200 ft long mortared rock masonry apron at the toe of the dam below the center spillway (U.S. Geological

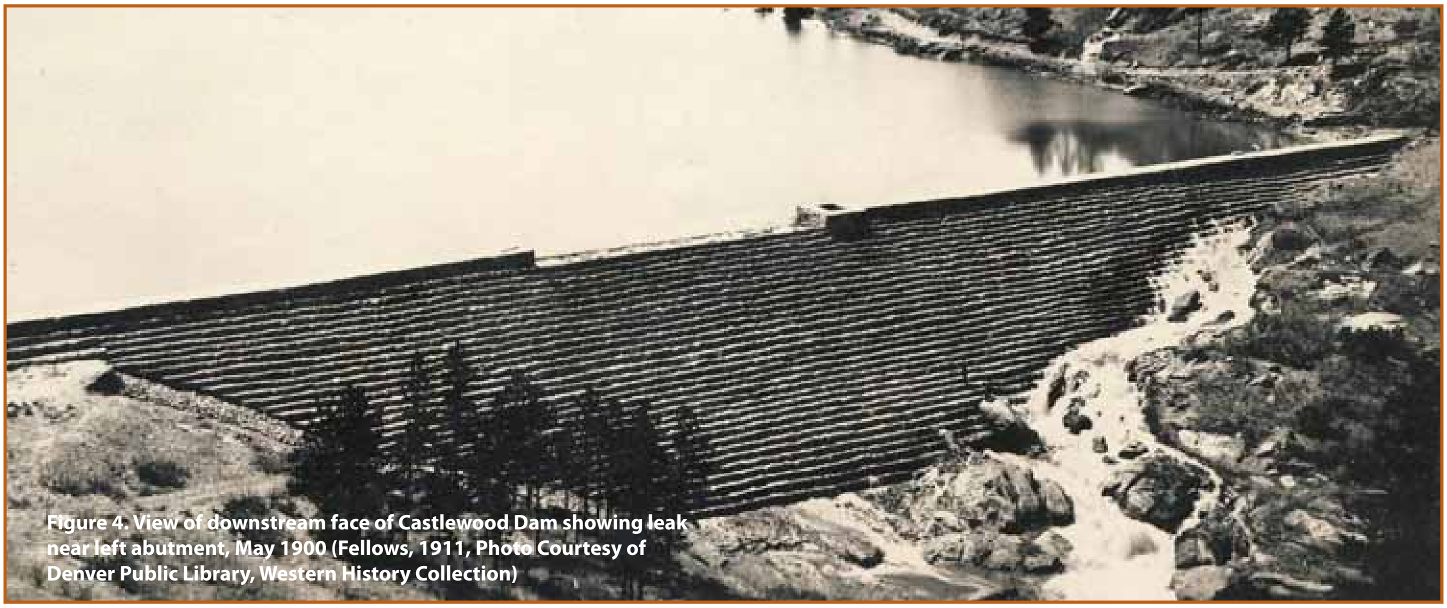

**Figure 4. View of downstream face of Castlewood Dam showing leak near left abutment, May 1900 (Fellows, 1911, Photo Courtesy of Denver Public Library, Western History Collection)**

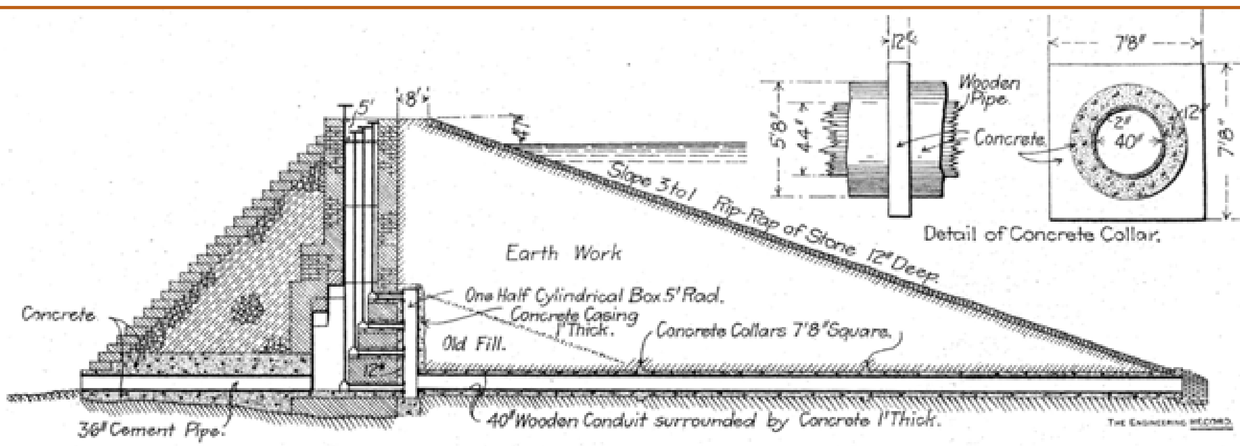


**Figure 5. Plans for 1902 reconstruction of the Castlewood Dam (Engineering Record, 1902, Reprinted Courtesy of Engineering News Record)**



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Survey, 1899). The masonry apron was reported to consist of closely laid rock from 3 to 6 ft in depth, which was then top-grouted and finished with cement mortar (Welles, 1898).

In mid to late April of 1900, high rainfall filled the reservoir to capacity. Newspaper reports indicate that at this time, large amounts of water were flowing through the dam, near a bulge in the lower left side of the downstream face. As evident in Figure 4, there also appears to be a substantial amount of water flowing along the dam/foundation contact near the middle of the center spillway. The highest reservoir level during this time period was reached on April 30, 1900. Exact reservoir levels for this event could not be found but it was reported that 500 cfs was flowing through the main spillway and bypass spillway for approximately 30 hours, causing substantial erosion of the bypass spillway on the left abutment (Engineering Record, 1900). This spillway erosion is still visible today.

After this event, the state engineer ordered that the reservoir be emptied until repairs had been made (Engineering Record, 1900). Following its purchase of the dam in approximately 1902, the Denver Sugar, Land and Irrigation Company, made repairs, which consisted of placing an upstream earthen embankment up to the base of the spillway at a 3:1 (H:V) slope protected by riprap, building a small masonry wall across the left abutment bypass spillway inlet so it would only flow when water was 1 ft above the crest of the dam or higher, building a new 12-ft wide unlined by-pass spillway near the right abutment, and reconstructing the inlet to the valve chamber. Figure 5 shows plans for the reconstruction approved by the state engineer.

After the 1902 repair, it appears that the dam functioned without significant problems until its failure in 1933.

After the 1902 repair, it appears that the dam functioned without significant problems until its failure in 1933. Based on statements made by the State Engineer and dam tender (who tended the dam for the last 20 years of its life), the dam was closely monitored during this time period. The following statements were made by the State Engineer and dam tender regarding observations made during the years prior to the dam's failure.

- The center spillway flowed periodically throughout the last 20 to 30 years of Castlewood Dam's life. The dam tender stated that he had at times observed discharges over the spillway to a depth of 2 ft for several hours duration with no observable damage to the dam or scour near the toe of the dam. The dam tender also made the following statement regarding the highest flow he had ever observed, which occurred in 1924: "Water flowed through the spillway in the dam to a maximum depth of 4 ft, but did not overtop the dam. At that time the discharge over the dam caused considerable vibration in the structure, but resulted in no visible damage, with the exception of causing erosion at the lower end of the spillway about four feet in depth."
- Although seepage was present along the toe of the dam, the flow was always clear and the amount of flow did not change substantially with reservoir level. The dam tender specifically noted that he had observed the flow on a regular basis in the weeks before the failure and did not see any changes in color or amount of flow.
- During the yearly inspections made by the State Engineer, no dangerous structural conditions were observed at the dam (Denver Post, August 9, 1933).

Records in the Colorado State Archives show that from spring 1914 to a few days before its failure in 1933 the reservoir was filled to the center spillway crest (4 ft below the dam crest) on an almost yearly basis for periods of time ranging from days to months.

## The Failure

According to eyewitness accounts and water levels in various containers left out overnight, the upper drainage basin of Cherry Creek received 4 to 9 inches of rain in approximately 3 hours on August 3, 1933. The only witness to the failure was the dam tender (Hugh Paine), who made the following statements regarding the failure and events leading up to it.

- At 11:15 PM the water level in the dam was 6 ft below the center spillway crest.
- After hearing a "rush of air of tornado proportions" the dam tender went out to the dam to make observations as he anticipated there would be a rush of water into the reservoir.
- At 12:00 AM the water had risen to the crest of the dam.
- By 12:15 AM "a torrent of water was pouring over and thru the dam, and within a few minutes the surface of the reservoir had dropped thirteen feet below the spillway." In a later statement it is indicated the depth of dam crest overtopping was over 1 ft.
- "The lapse of time between the inrush of water into the reservoir and the time of the dam failure did not exceed 45 minutes." (Denver Post, August 9, 1933)

After witnessing the dam failing, the dam tender left the site to warn people downstream that the dam had broken and a flood was rushing downstream. The first wave of the flood reached Denver (35 miles

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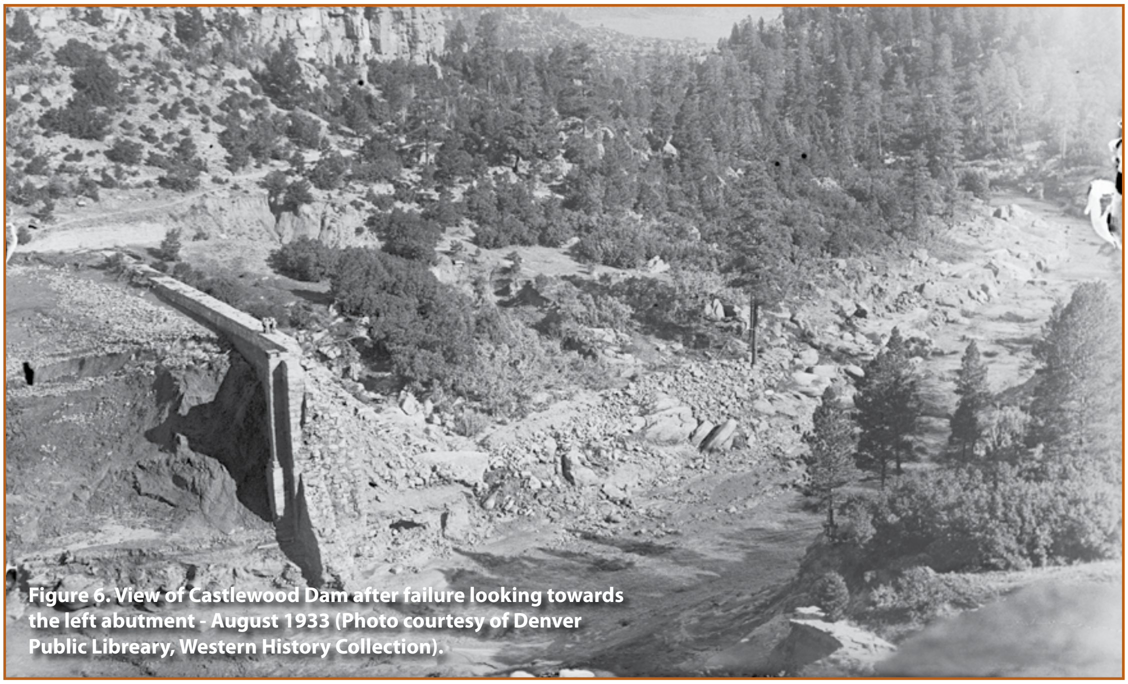

Figure 6. View of Castlewood Dam after failure looking towards the left abutment - August 1933 (Photo courtesy of Denver Public Library, Western History Collection).

downstream) at 5:40 am with waves 3 to 6 ft high, attributed to the initial high spillway discharge and overtopping. The second wave of the flood reached Denver at 6:20 am with a wave approximately 15 ft high, attributed to the dam breaching (Houk, 1933). Cherry Creek flows through the Denver Country Club area, which is still one of the wealthiest parts of Denver, and along Speer Blvd through the heart of downtown Denver. This area is now protected by the U.S. Army Corps of Engineers Cherry Creek Dam.

Photos taken the day after the failure and descriptions of the failure indicate that almost the entire right half of the dam had washed away, including the center spillway (Figure 6). It appears from these photos that erosion occurred to just below the base of the foundation contact near the upstream wall of the dam and to a somewhat lower elevation near the downstream wall of the dam.

## Forensic Investigations

Reports published by the State Engineer and a number of engineering journals stated that the exact cause of the failure could not be known. However, they generally agreed that failure was likely a result of overtopping flow either scouring the toe of the dam and undermining the masonry wall, or scouring of the downstream face of the dam. The fact that the Castlewood Dam failed during an overtopping event does not necessarily mean that the overtopping itself was the primary cause of failure. There have been cases where even though a dam failed during an overtopping event, the post failure analysis revealed that the overtopping was either not the primary cause of failure or that other factors also played a significant role. This is due to the fact that during an overtopping event a dam is typically subjected to higher loads than it receives during regular operation. These additional loads can cause foundation or structural shear failures.

The additional head can also lead to the development of a critical gradient and the initiation of piping erosion along the foundation contact or within the foundation for masonry dams founded on weak material. Although there is some evidence indicating that piping erosion and foundation or structural shear deformation may have been problematic, particularly in the first 12 years of the dam's life, it appears that these problems only contributed to failure by possibly weakening the dam rather than directly causing its failure. The large embankment placed during the reconstruction in 1902 also likely reduced the potential seepage issues.

Based on the historical reservoir levels for the Castlewood Dam, it appears that the dam was regularly subjected to loads near the crest of the dam for at least 20 years prior to its failure. The reservoir level during the overtopping event was only a little over 1 ft higher than the dam had been subjected to in the past and only 3 to 5 ft higher than spring reservoir levels that occurred on an almost yearly basis for up to months at a time. Considering that the dam had survived over 30 years experiencing loads of similar magnitude to those it received during failure with no observable signs of distress, it does not seem likely that causes other than scour erosion from the overtopping were the primary cause of failure. If there were other primary causes of failure, they would have likely failed the dam earlier in its life rather than coincidentally during the 45 minutes that the dam was being overtopped.

Our recent investigations were aimed at trying to verify that scour would be predicted under the conditions of the dam's failure and gaining a better understanding of where the failure likely initiated through modern quantitative evaluation methods. The Stream Power – Erodibility Index method (Annandale, 2006) was chosen for the evaluation due to its simplicity relative to the available information.

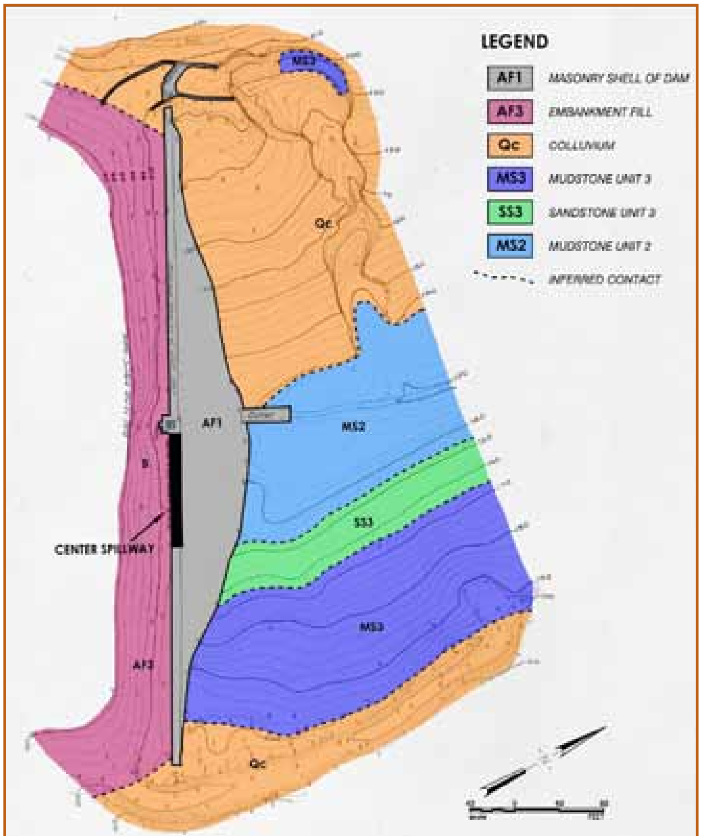


**Figure 7. Geologic plan map showing recently mapped contacts**

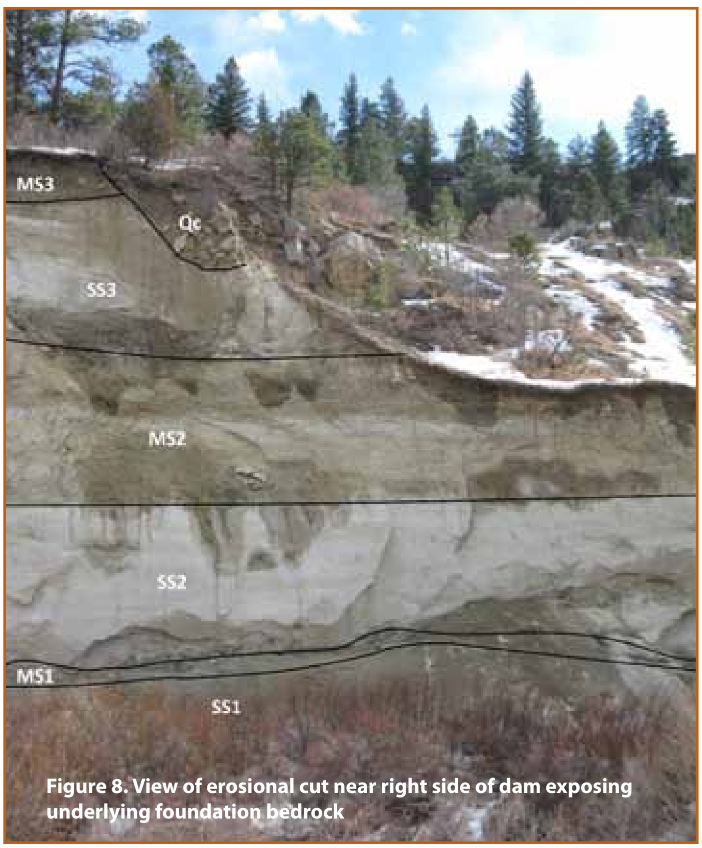


**Figure 8. View of erosional cut near right side of dam exposing underlying foundation bedrock**

## Geological and Geotechnical Characterization

The first step in the analysis was to perform geological and geotechnical characterizations of the foundation rock and the masonry. Figure 7 shows a geologic map developed from the available information, and Figure 8 shows the sandstone and mudstone rock units exposed in the erosional cut on the right side of the canyon. The dam was founded on the MS3 mudstone, the SS3 sandstone, and the MS2 mudstone (although the left abutment of the dam appears to be founded on large colluvial boulders derived from the conglomerate cap rock). Therefore, the canyon has eroded more than 20 feet below the original base of the dam since its failure.

Table 1 provides a brief description of these units and the dam facing/masonry apron. Although some of the units possess borderline soil properties, the Erodibility Index based on rock characterization is summarized in Table 2. The Erodibility Index, K, is the product of Ms, Kb, Kd, and Js. The other parameters listed in Table 2 are used to derive some of these values using methods described in Annandale (2006). Due to considerable uncertainty in these values, high and low values were considered. For simplicity, only average values are summarized in Table 2. The use of Erodibility Index will be described later in this article.

### Table 1 – Summary Material Descriptions

| UNIT | BRIEF DESCRIPTION |
|---|---|
| **Masonry** | Hard conglomerate blocks with minor surface weathering cemented together with mortar. |
| **MS3** | Dark bluish gray siltstone with trace of fine sand. Fresh to slightly weathered. Generally massive with closely to very closely spaced (0.5 to 3 inches) tight fractures. Fracture walls are slightly rough to rough and planar with some iron staining. Breaks into square fragments approximately ½ inch to 3 inches in diameter with moderate hammer blows. |
| **SS3** | Friable, pinkish-tan, fine to coarse grained sandstone. Fresh to slightly weathered. Variable – in many areas it can be crumbled by hand with only light to moderate pressure – in some areas moderate blows with a rock hammer were required to dislodge samples and the samples could not be crumbled by hand. Difficult to sample. |
| **MS2** | Dark olive brown sandy siltstone. Moderately weathered. Massive with very closely spaced (0.5 to 2 inches) tight fractures. Fracture walls are slightly rough to rough and planar with some iron staining. Breaks into square fragments approximately ½ inch to 2 inches in diameter with light hammer blows. |


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**Table 2 – Average Erosion Resistance of Rock.** UCS = unconfined compressive strength, $M_s$ = mass strength number, RQD = rock quality designation, $J_n$ = joint set number, $K_b$ = block size number, $J_r$ = joint roughness number, $J_a$ = joint alteration number, $K_d$ = inter-block shear strength number, $J_s$ = shape and orientation number, K = erodibility index. Larger values of erodibility index indicate increased resistance to erosion.

| Material | UCS (MPa) | $M_s$ | RQD | $J_n$ | $K_b$ | $J_r$ | $J_a$ | $K_d$ | $J_s$ | $K$ |
|---|---|---|---|---|---|---|---|---|---|---|
| **Masonry** | 30 | 30 | 100 | 2.73 | 36.6 | 3 | 1 | 3 | 1 | 3300 |
| **MS3** | 2.5 | 2.0 | 53 | 1.83 | 28.7 | 2 | 1 | 2 | 0.5 | 59 |
| **SS3** | <1.7 | 0.87 | 53 | 1 | 53 | 4 | 1 | 4 | 1 | 183 |
| **MS2** | <1.7 | 0.87 | 53 | 1.83 | 28.7 | 2 | 1 | 2 | 0.5 | 25 |

## Stream Power Evaluation

In evaluating the erosive power of the water going over the dam, it had to be determined whether the water flowing over the stepped downstream face would be nappe flow, where the flow follows and energy is dissipated by the steps, or skimming flow, where the flow is deep enough that most of the water skims over the steps and much less energy is dissipated. A depth of flow of about 1.53 feet over the center spillway was calculated as the approximate transition point between nappe and skimming flow using the method described in Boes and Hagar (2003). Since the dam had experienced many instances of up to 2 feet of flow depth over the spillway with no evidence of distress, only skimming flow was considered in this study. A second consideration related to the amount of tailwater that might be present to dissipate energy at the toe of the structure. A discharge curve for water flowing over the spillways and eventually the dam crest was developed. Using this curve and the best available pre-failure topography, a HEC-RAS model was used to assess potential steady state tailwater conditions. Since the storm that overtopped the dam occurred suddenly, it was uncertain as to whether steady state tailwater would have developed, and both steady state tailwater and no tailwater conditions were evaluated. The methodologies described by Boes and Hager (2003) and Annandale (2006) were combined to calculate the Stream Power at the toe of the dam from the overflowing water. For a water surface above the crest of the dam, energy would be dissipated by nappe flow over the abutment sections, but water would flow down the groin of the dam with increasing Stream Power as it moved toward the channel. For areas above tailwater, the Stream Power from water flowing down the groin was added to the Stream Power of the overtopping flows, assuming a reasonable width of flow and an energy line equal to the slope of the groin. The results of the Stream Power evaluation are summarized in Tables 3 and 4.

## Scour Prediction

For this investigation, probability of scour was assessed by using a logistic regression developed by Wibowo, et. al. (2005) given by Equation 1. This analysis was completed for the central toe area of the dam using both the no tailwater scenario and the steady state tailwater scenario, and the groins of the dam for the no tailwater scenario.

The results of this evaluation at the toe of the dam are summarized in Table 3 and Figure 9. Only the masonry and MS2 units would be impacted at the toe of the dam.

$$P(E) = \frac{1}{1 + \exp\left[-\left(-1.859 - 7.029 \cdot \log(K) + 9.798 \cdot \log(SP)\right)\right]} \quad (1)$$

Where: $P(E)$ = probability of scour initiation.
$K$ = erodibility index.
$SP$ = stream power (kW/m$^2$).

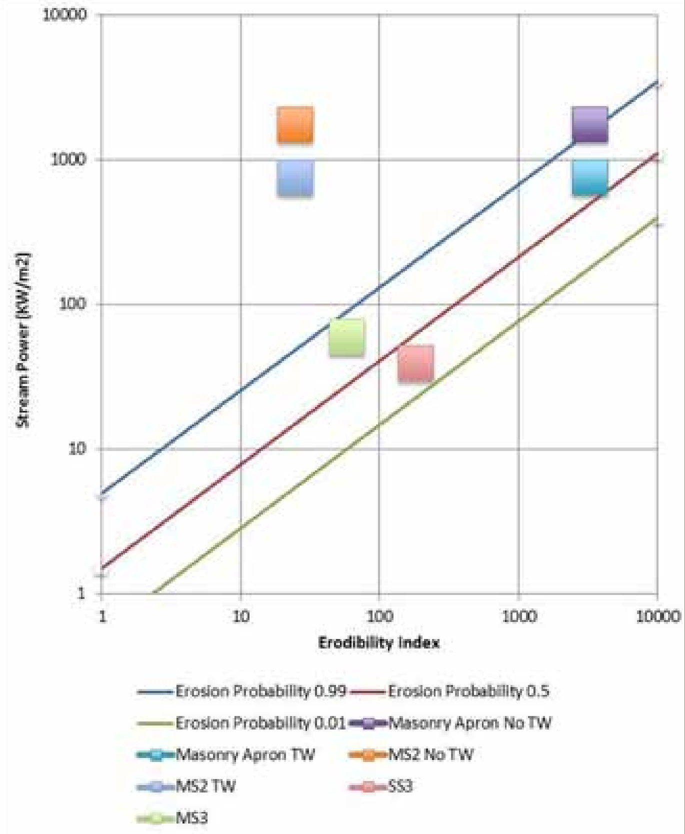


**Figure 9. Scour prediction results for maximum overtopping conditions at failure**

**Table 3 – Results of Scour Analysis at Toe of Dam**

| Height Above Spillway Crest (ft) | Corresponds to | Tailwater Depth (ft) | Stream Power (kW/m$^2$) | Masonry at Toe Probability of Erosion | MS2 Probability of Erosion |
|---|---|---|---|---|---|
| 1.53 | Threshold for skimming flow | 0 | 330 | 0.13 | ~0.999 |
| 2 | 1902-1933 max level achieved several times | 0 | 482 | 0.42 | ~0.999 |
| 4 | 1924 flood event resulted in 4 ft deep scour hole | 0 | 1171 | 0.97 | ~0.999 |
| 5.34 | Estimated depth at time of failure - Aug 3, 1933 | 0 | 1755 | 0.99 | ~0.999 |
| 1.53 | Threshold for skimming flow | 2.1 | 286 | 0.07 | ~0.999 |
| 2 | 1902-1933 max level achieved several times | 3.5 | 367 | 0.18 | ~0.999 |
| 4 | 1924 flood event resulted in 4 ft deep scour hole | 9.2 | 756 | 0.83 | ~0.999 |
| 5.34 | Estimated depth at time of failure - Aug 3, 1933 | 16.5 | 692 | 0.77 | ~0.999 |


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**Table 4 – Results of Scour Analysis along the Groins at Maximum Overtopping**

| Groin Material | Estimated Stream Power (kW/m$^2$) | Erodibility Index | Probability of Erosion |
|---|---|---|---|
| MS3 | 39 | 59 | 0.78 |
| SS3 | 60 | 183 | 0.41 |

Even though direct photographic evidence of the masonry apron could not be found, the results are consistent with the reported behavior given a masonry apron in place. For flows over the central spillway less than 2 ft in depth, the analysis predicts erosion would be unlikely, especially if some tailwater was to build up (probability of erosion 0.18 to 0.42). Erosion of the masonry apron would be likely with 4 ft of flow over the central spillway, consistent with the erosion hole that formed. At the maximum overtopping depth, prior to failure, erosion is virtually certain, especially before tailwater had a chance to build. If the masonry apron is lost, erosion of the underlying MS2 is predicted to occur rapidly. An evaluation of stream power at elevations below the toe of the dam indicates that scour depths of over 20 ft would be probable for the overtopping event.

At maximum overtopping, flow down the groins would potentially erode the MS3 and SS3 units on the abutments. Table 4 shows an evaluation of the average potential for erosion at the point of maximum discharge along the groin. It can be seen that the upper more erodible MS3 has a higher likelihood of erosion than the lower SS3 unit even though the Stream Power at overtopping is less along the upper groin. In both cases, it appears that the likelihood of erosion along the groins is less than that at the toe of the dam, even with the masonry apron in place. This can be seen in Figure 9 where the MS3 point is closer to the 0.99 probability line than the SS3 point.

## Empirical Evidence

The most significant piece of evidence supporting an overtopping erosion failure mode discovered during the site investigation was the existence of weak, highly erodible sandstones and mudstones underlying the remains of the dam. It is readily apparent based on visual inspection alone that this bedrock would easily scour. The more than 20 ft of scour erosion that has occurred in this canyon since the dam's failure also suggests that the mudstone and sandstone bedrock is highly erodible.

The erosion that occurred from wave action early in the dam's existence suggests the upper abutment rock is highly erodible. Similarly, the substantial scour that is evident in the left bypass spillway provides strong qualitative evidence of the highly erodible nature of at least some of the materials on site. The majority of the scour observed in this spillway was reported to have occurred during a 500 cfs flow event over the course of 30 hours in April, 1900. While not enough information was present for this event to obtain accurate stream power estimates, the relatively low flow rate reported suggests that flow depth was probably about 2 ft at most. Given the low flow rate along with the relatively shallow slope of the left bypass spillway that was present before the erosion, the Stream Power was likely low. It follows then that the erosion resistance of the material was also low. Inspection suggests this erosion extended into the MS3 unit, although the extent of erosion into the MS3 was unclear due to slopewash that has accumulated over the years.

Reports of no scour at 2 feet of flow over the central spillway, and a 4-foot-deep scour hole at 4 feet of flow over the spillway provides an indication of the erodiblity of the materials at the downstream toe of the dam and confirms that the masonry apron described in the records probably existed. The left portion of the dam survived. Inspection indicates that this portion of the dam was actually founded on large conglomerate colluvial boulders. This material would be resistant to erosion and would protect that portion of the dam. It should be noted that the portion of the dam that survived is the portion that leaked the most; this portion exhibits a bulge on the downstream face. Thus, the bulge was not indicative of a critical structural condition.

## Conclusions

The historical records research and literature review generally showed that the dam had been subject to numerous high reservoir level loading events prior to failure, that the dam did in fact overtop over the entire length of the crest during the 1933 thunderstorm flood, and that there was no significant evidence to support any failure mode other than overtopping and scour of either the downstream masonry of the dam or the materials near the toe. The Erodibility Index Method for scour analysis provided results entirely consistent with the observed behavior and an overtopping erosion failure. Based on this evaluation, the operative failure mode was most likely as follows: During the 1933 thunderstorm the reservoir rapidly filled until water flowed over the central spillway and eventually overtopped the remainder of the dam by a little over a foot. Prior to the build-up of tailwater, erosion likely initiated at the toe of the central spillway section. The masonry apron in this location was washed away and deep erosion of the underlying mudstone quickly ensued. At the same time, scour may have begun along the groins due to the overtopping flow. Support for the mortared downstream masonry facing was lost, and it collapsed into the erosion hole. The interior rubble masonry progressively collapsed and eroded. This likely created a hole under the dam that caused the reservoir to drop rapidly as observed by the dam tender. Eventually, the mortared upstream masonry face was exposed over a large enough area that it could no longer support the reservoir load and collapsed. This sent a sudden wall of water downstream, scattering large pieces of masonry along the channel and causing downstream damages all the way through Denver. The flood wave resulted in two deaths and over $1,000,000 (1933) in damage. And that is why Castlewood was not worth a dam.


## Acknowledgement

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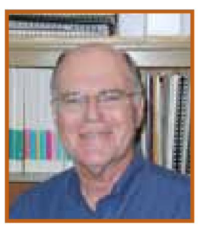


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