Reassessment of the Castlewood Dam Failure

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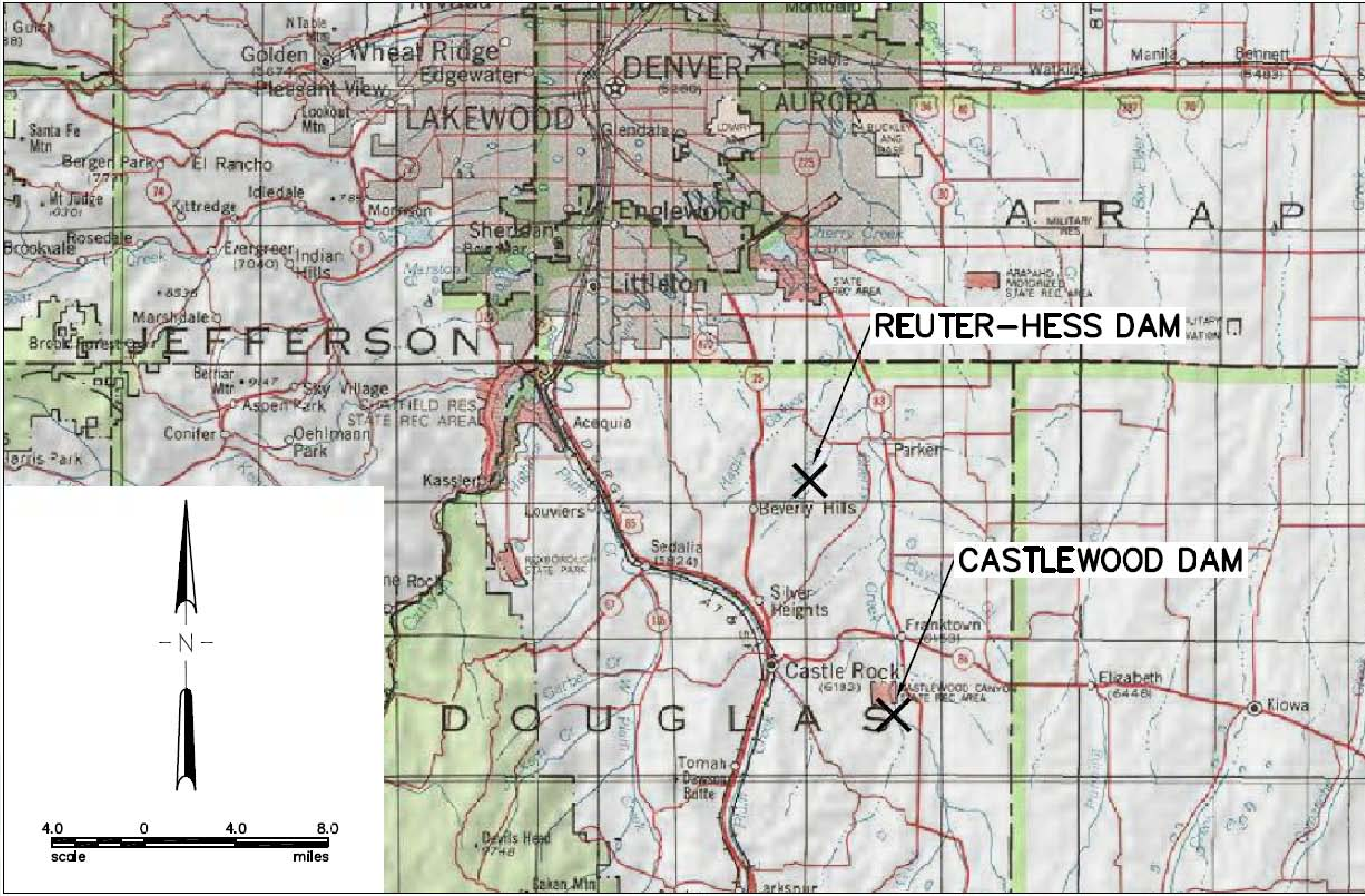
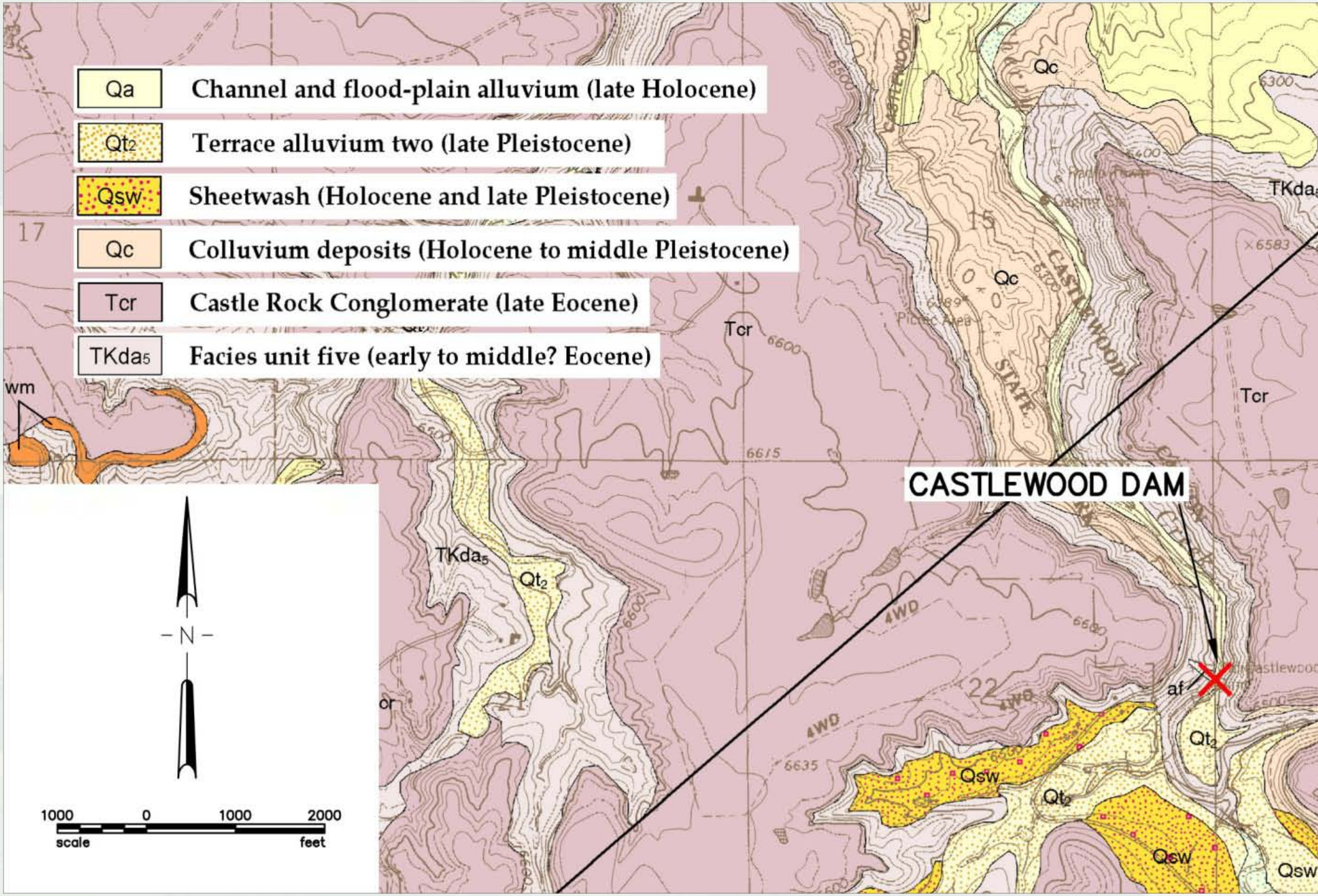
Introduction

- Masonry rock-fill dam constructed in 1890 across Cherry Creek south of Denver, Colorado.
- Failed at ~12:15 am on August 3rd, 1933 after a heavy rainfall.
- Flood resulting from the dam break flowed through downtown Denver causing over $1,000,000 (1933 dollars) in damage and two deaths.


REUTER–HESS DAM
CASTLEWOOD DAM
DENVER
AURORA
LAKEWOOD
Golden
Wheat Ridge
Edgewater
Englewood
Littleton
Sheridan
JEFFERSON
DOUGLAS
Parker
Beverly Hills
Castle Rock
Franktown
Elizabeth
Kiowa
Sedalia
Louviers
Silver Heights
Conifer
N
4.0
0
4.0
8.0
scale
miles

Regional Geology

- Castlewood Dam is located near the western edge of the Denver Basin.
- Area generally consists of Paleocene and Eocene age (50-65 million years ago) bedrock overlain by thin layers of Quaternary alluvium and colluvium.
- Bedrock consists of:
  - Castle Rock Conglomerate.
    - Horizontal to shallow dipping pebble, cobble, and boulder arkosic conglomerate.   Fairly strong cap-rock material forming steep cliffs.
  - Upper Dawson Formation.
    - Horizontal to shallow dipping beds of friable mudstones and sandstones.



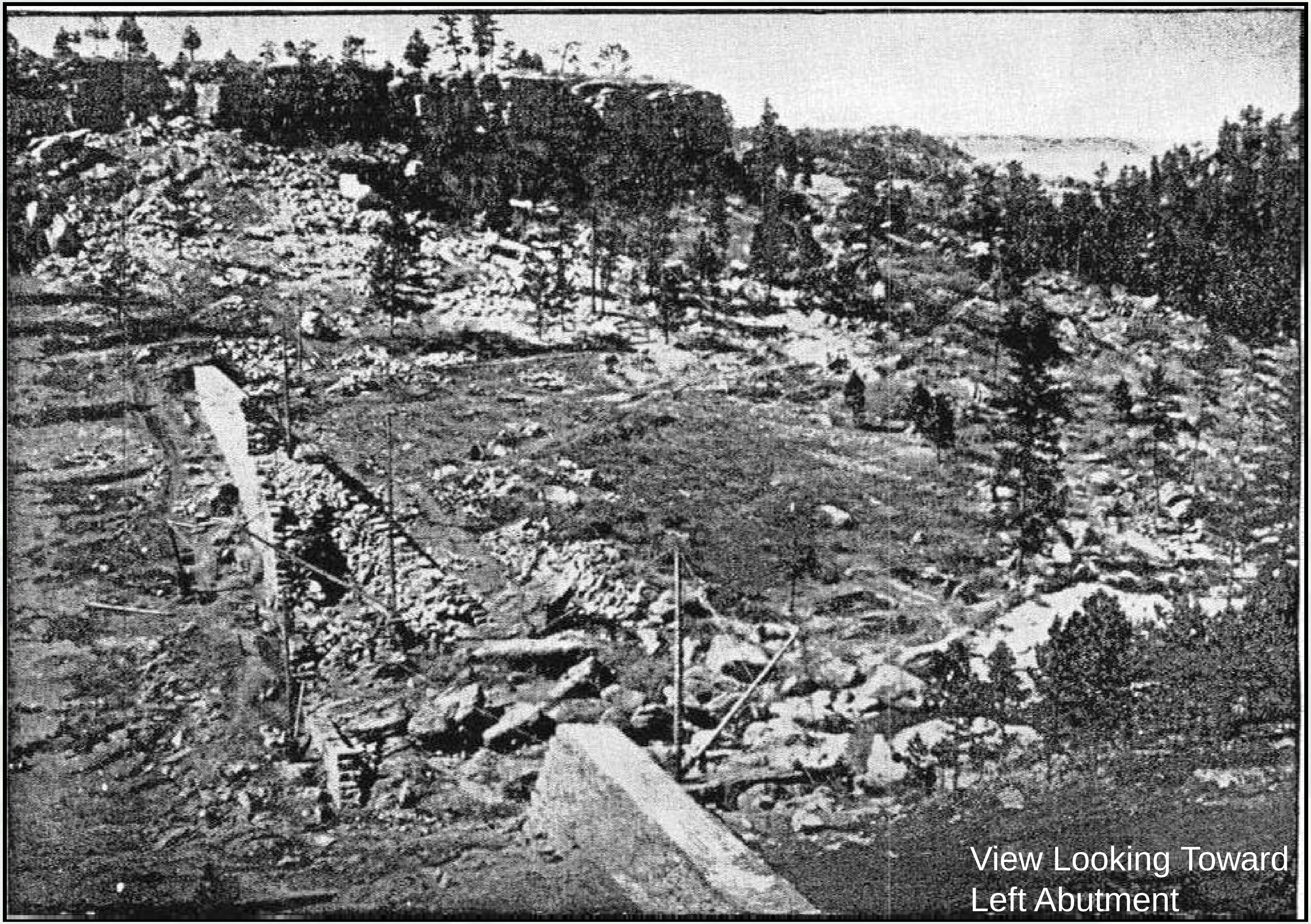
(Modified from Thorson, 2004)

Initial Construction

- Idea of constructing Castlewood Dam first raised in early 1889.
- Shortly thereafter, the Denver Land and Water Storage Company was formed and plans to build the dam were initiated.
- During the summer of 1889, plans for the dam were prepared by A.M. Welles.
- Proposed dam generally consisted of rubble fill between mortared masonry walls (not typical cyclopean masonry construction).
- Construction commenced in December 1889 and continued until completion in November 1890.
- Little documentation of original design intent and construction exists.
- Plan drawings of the dam not released until 1898 (after some modifications to the dam).

(Horan, 1997)

View Looking Toward Right Abutment

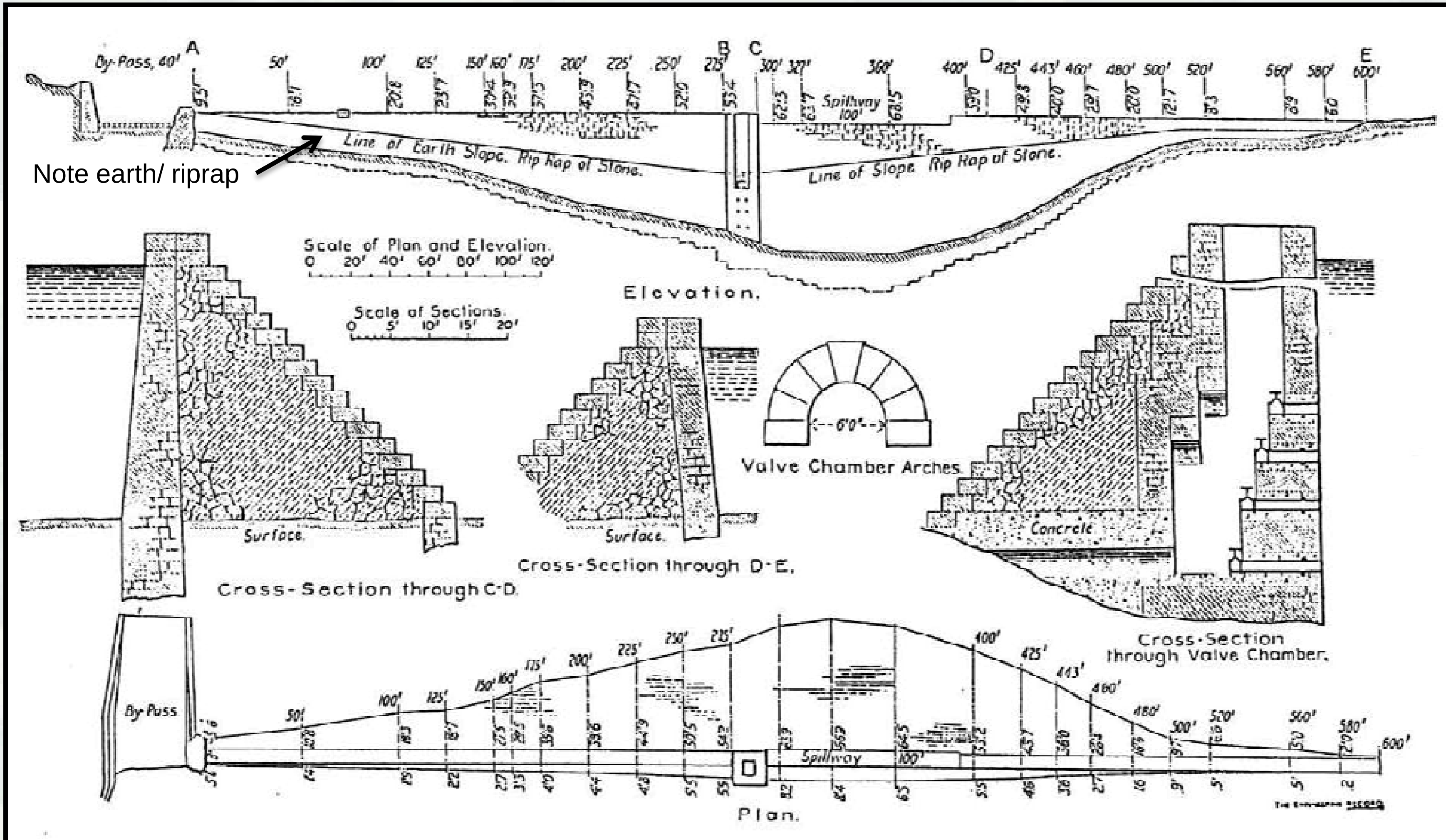


(U.S. Geological Survey, 1892)

Design Details as Reported

- 600 ft long.
- 70 ft high from reservoir floor.
- 92 ft high above lowest point of the upstream masonry wall foundation.
- 8 ft wide at crest.
- 100 ft long, 4 ft deep uncontrolled center spillway.
- 40 ft wide masonry lined bypass spillway near left abutment.
- Upstream masonry wall of Castle Rock conglomerate blocks set in mortar at 1:10 (H:V) slope founded on “hard sandy clay” (mudstone in Dawson Formation) 6 to 22 ft below ground surface.
- Downstream masonry wall of conglomerate blocks layered and mortared with 2 by 2 ft steps at a 1:1 slope founded on an unspecified foundation 10 feet below the ground surface.
- Rubble fill of various size rock placed directly on ground surface.
- Outlet structure with eight 12 inch inlet pipes and one 36 inch outlet Pipe.

Plans Published 8 Years After Construction



(Welles, 1898)

Performance History

- November, 1890 – May, 1897
  - Immediately after construction the dam showed signs of minor settlement and cracking along with seepage through the dam and along the foundation contact.
  - In the spring of 1891, an engineering committee inspected the dam and determined that the dam was faulty and improvements needed to be made.
  - Over the next few years the dam was operated with little maintenance.
  - No complete pool elevation records exist for this time period.
    - Scattered reports indicate the reservoir was rarely filled due to leakage.
    - Greatest reservoir depth reported was 8 inches above the center spillway crest.

Performance History

- May, 1897
  - Partial failure of the dam occurred due to waves washing against the u/s toe of the upper 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.

Performance History

- August, 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.
  - Reports also indicated that after this event a large crack ran up the dam at the west (left) end of the undermined area.

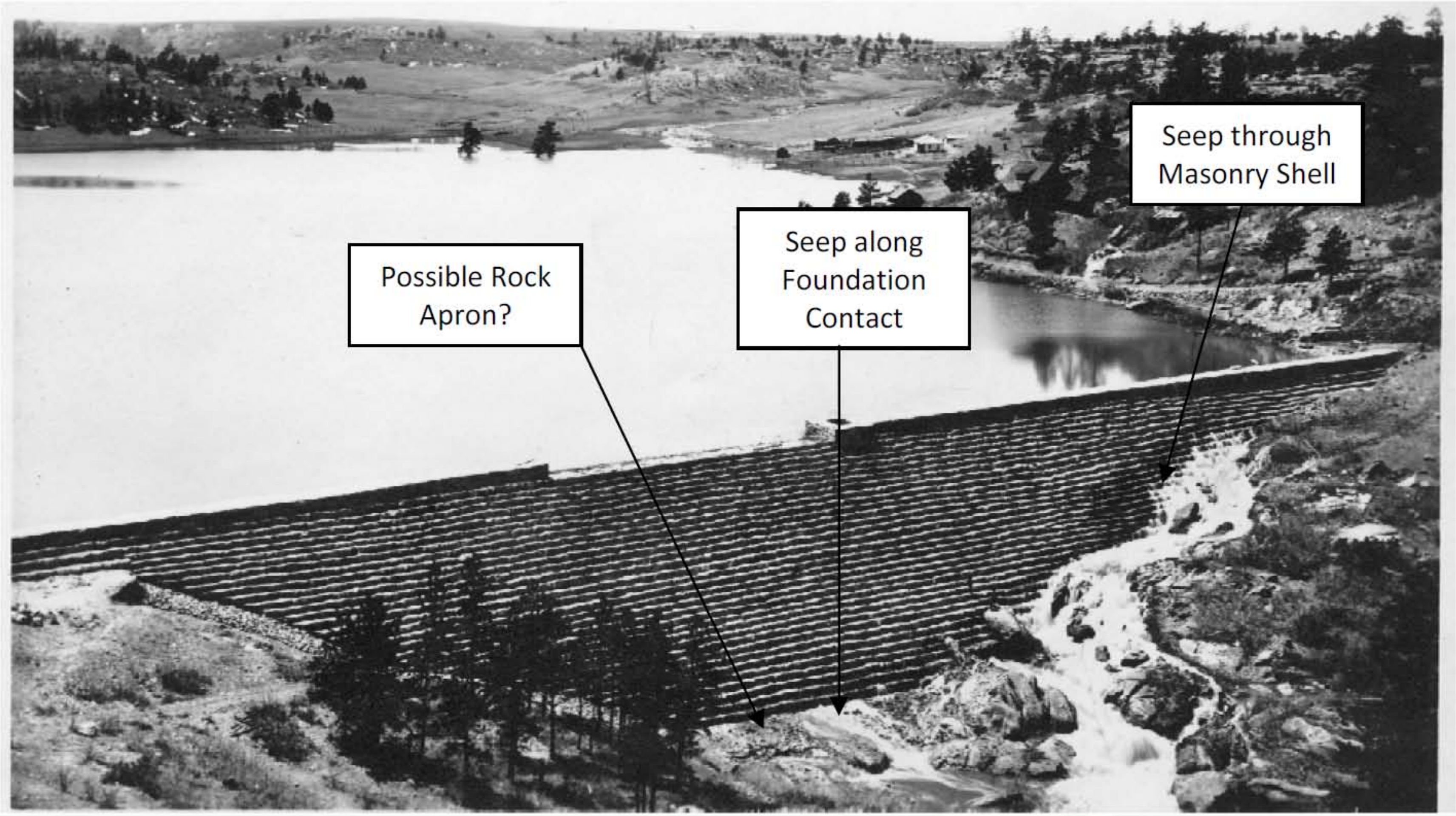
Performance History

- Summer, 1898 – April, 1900
  - In the Summer of 1898, repairs to fix the 1897 failure were made.
    - Heavily cracked portions of masonry were removed and replaced.
    - Clay puddle wall placed against the upstream masonry wall.
    - Upstream embankment raised.
    - 25 ft wide, 200 ft long mortared rock apron placed at the toe below the center spillway (reported as 3 to 6 ft deep closely laid rock that was top grouted).
  - After the dam was repaired, the leakage was still present but to a much lesser degree.
  - The reservoir was not filled in 1899.

Performance History

- April, 1900 – May, 1902
  - In mid to late April, 1900, heavy rainfall filled the reservoir to capacity.
  - Large amounts of water were flowing through the dam during this event.
    - Flow attributed to water entering cracks in the upstream masonry wall near the crest.
    - A later report also suggested that it may have been due to breaks in the iron inlet pipes.
  - Highest reservoir level during this event was reached on April 30, 1900.
    - Exact reservoir levels not known but reports indicated 500 cfs was flowing through the main spillway and bypass spillway for 30 hours.
    - Substantial erosion of the left bypass spillway occurred.

Photo Showing Leakage in May, 1900



(Fellows, 1911)

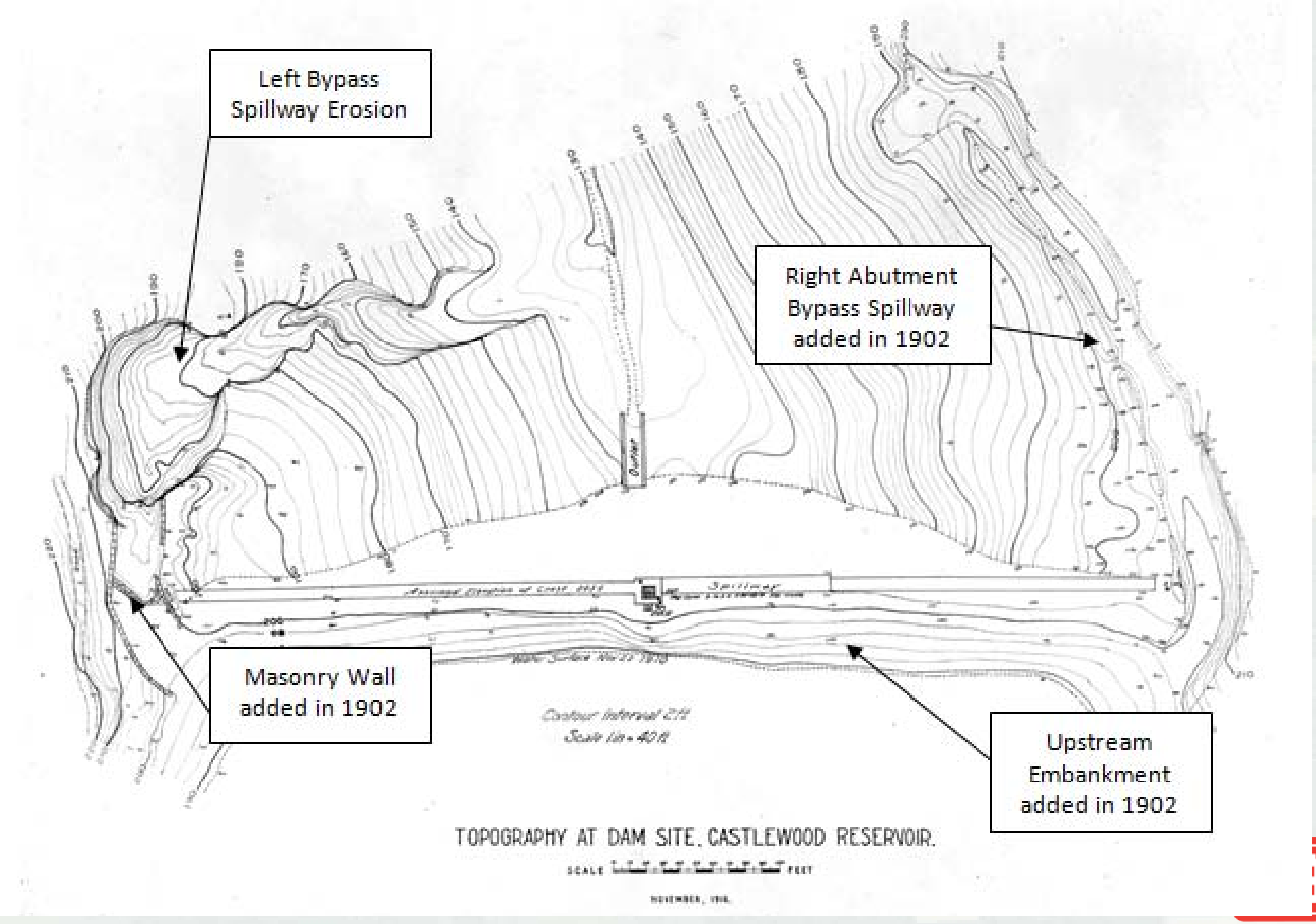
Photo Showing Leakage in May, 1900

(Photo courtesy of History Colorado)

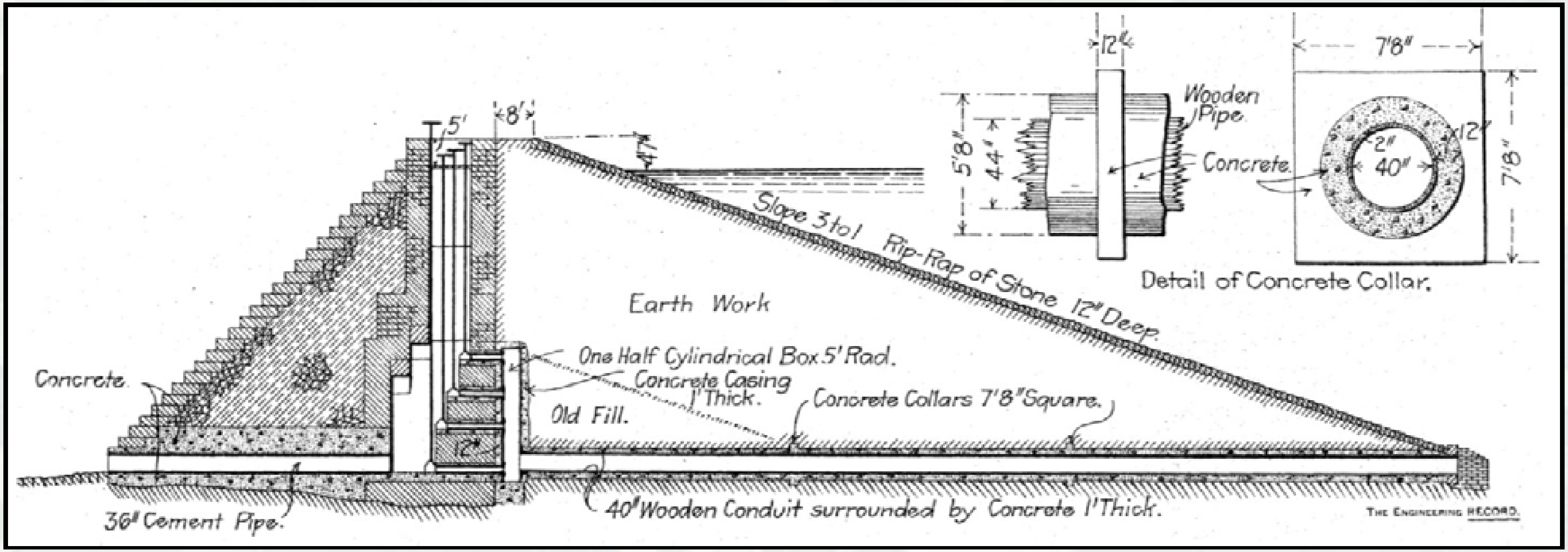
Performance History

- May, 1902 – July, 1902
  - Repairs and improvements made.
  - Upstream earthen embankment placed up to the base of the spillway.
    - Placed at 3:1 (H:V) slope.
    - Material and compaction unspecified.
    - Rip-rap protection.
  - Small masonry wall built at entrance to left bypass spillway to a height of 1 ft above the crest of the dam.
  - 12 ft wide unlined by-pass spillway constructed near right abutment.
  - Inlet reconstructed.

Topographic Map of Site from 1910



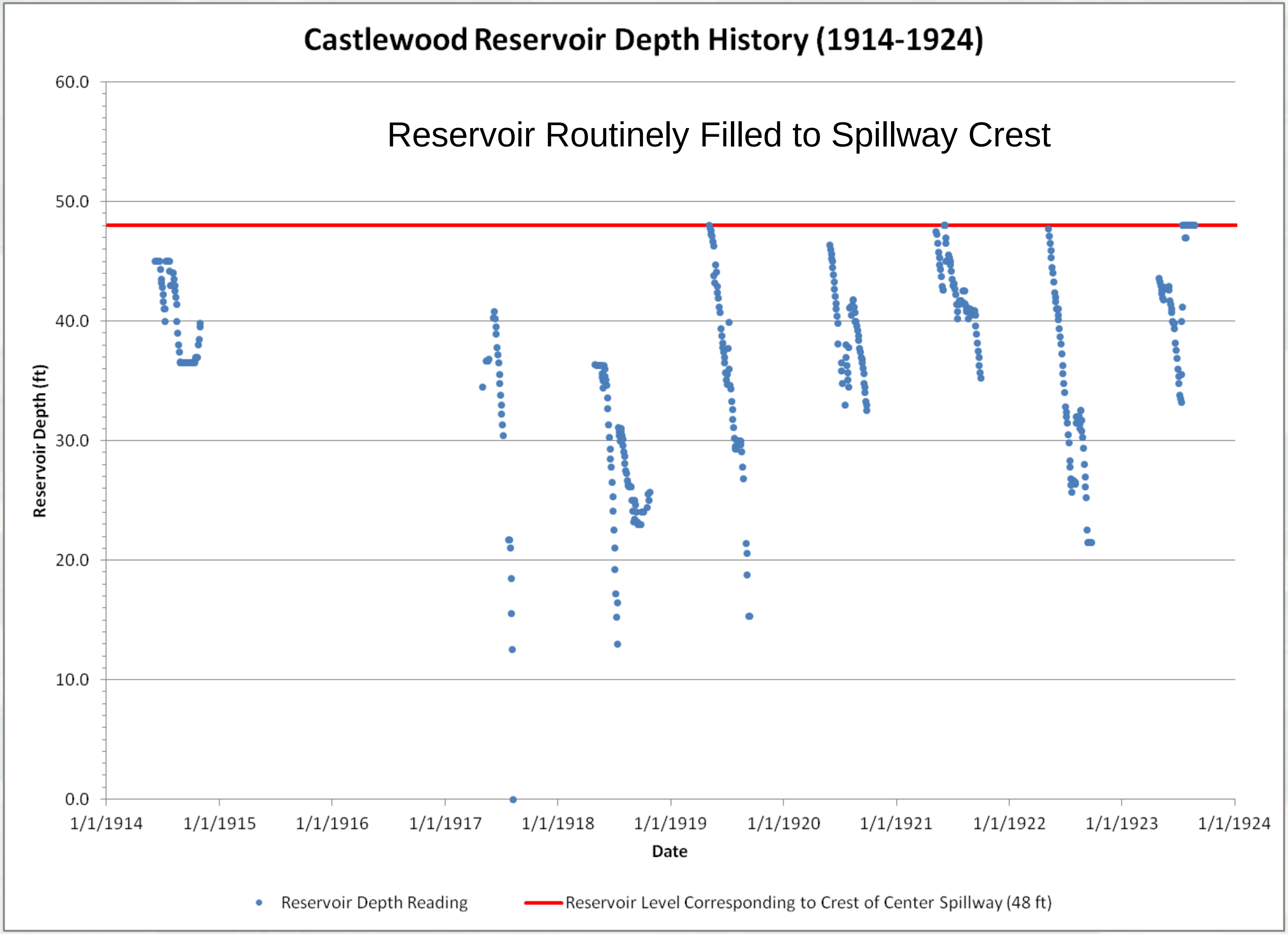
Plans for 1902 Reconstruction

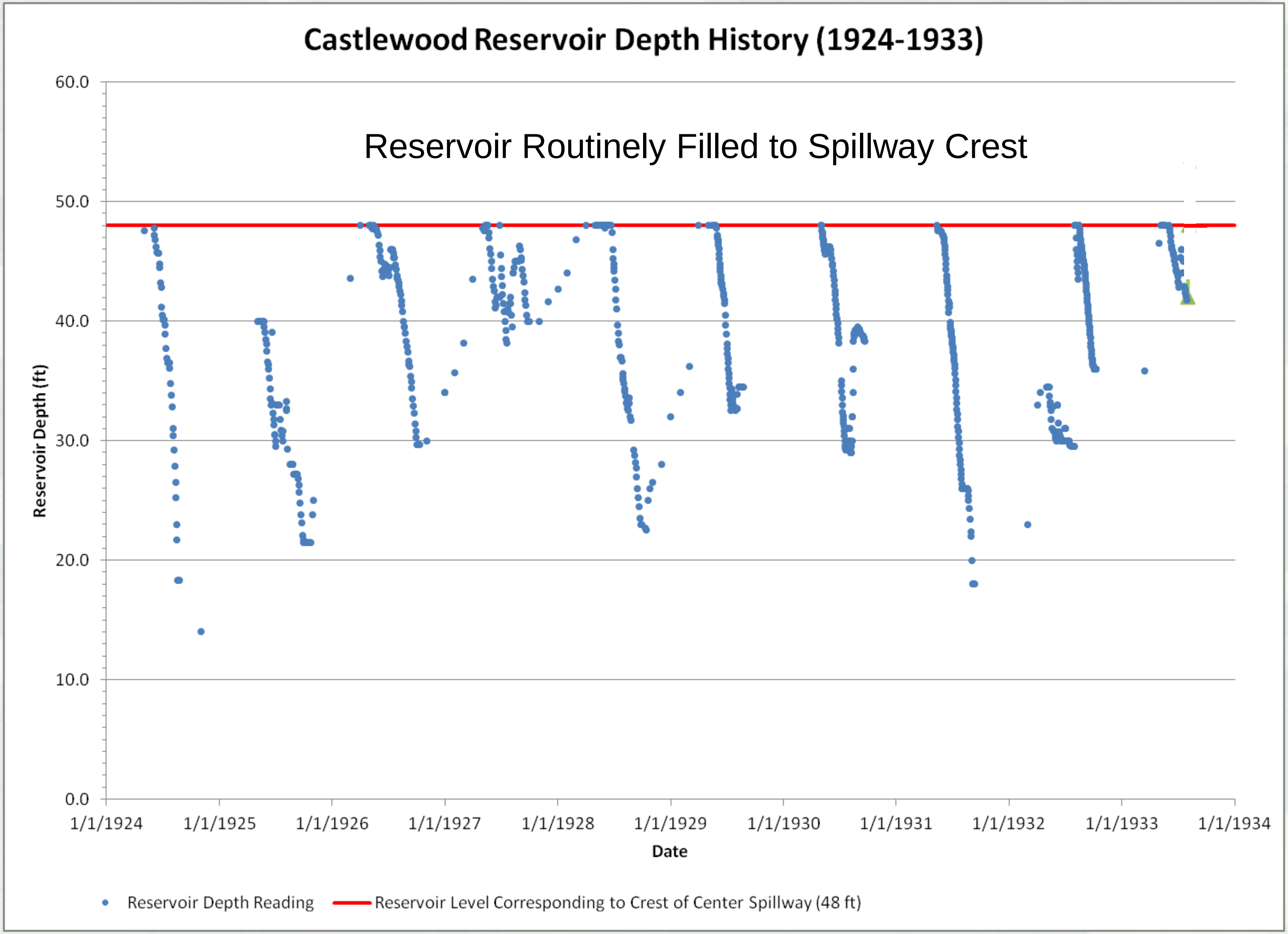


(Engineering Record, 1902)

Performance History

- July, 1902 – August 2, 1933
  - After the 1902 reconstruction, the dam functioned without reported significant problems until its failure.
  - The following statements were made by the State Engineer and the dam tender regarding observations of the dam prior to failure.
    - Center spillway flowed periodically throughout the last 20-30 years of the dam's operation.
    - Spillway flow depths of 2 ft for several hours observed with no damage to dam or downstream toe.
    - 1924, 4 ft spillway flow event caused a scour hole at the toe 4 ft in depth. No other damage observed.
    - Some seepage present along the toe but flows was always clear and amount of flow did not change with reservoir level.
    - No dangerous structural conditions observed in yearly inspections
  - Reservoir level records discovered in the Colorado State Archives from spring 1914 to a few days before failure.


Castlewood Reservoir Depth History (1914-1924)
Reservoir Routinely Filled to Spillway Crest
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Reservoir Depth (ft)
1/1/1914
1/1/1915
1/1/1916
1/1/1917
1/1/1918
1/1/1919
1/1/1920
1/1/1921
1/1/1922
1/1/1923
1/1/1924
Date
Reservoir Depth Reading
Reservoir Level Corresponding to Crest of Center Spillway (48 ft)


Castlewood Reservoir Depth History (1924-1933)
Reservoir Routinely Filled to Spillway Crest
60.0
50.0
40.0
30.0
20.0
10.0
0.0
Reservoir Depth (ft)
1/1/1924
1/1/1925
1/1/1926
1/1/1927
1/1/1928
1/1/1929
1/1/1930
1/1/1931
1/1/1932
1/1/1933
1/1/1934
Date
Reservoir Depth Reading
Reservoir Level Corresponding to Crest of Center Spillway (48 ft)

Dam Site Characterization

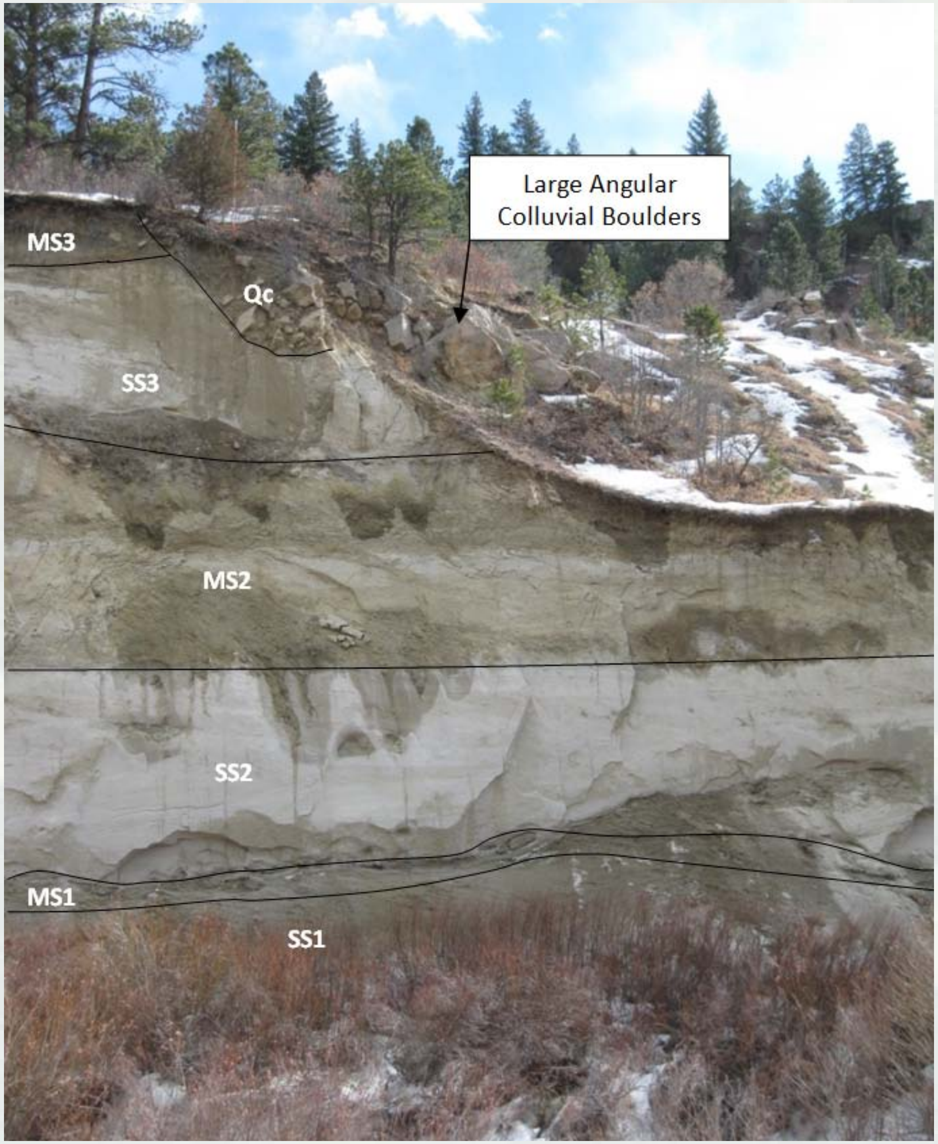
- Site characterization consisted of:
  - Photogrammetry (by USBR).
  - Characterization of the site geology.
    - Elevations of bedrock contacts used in conjunction with pre-failure topo to create pre-failure geologic maps and cross sections.
  - Engineering geologic evaluations of site materials.
    - Surficial observations – Drilling not possible in State Park.
    - Schmidt hammer.
    - Point load testing.
    - Correlations to drilling at Reuter-Hess Dam (12 miles NE of Castlewood Dam site in same formation).
    - Weak-rock classification including jar slake tests.
  - Visual inspection of dam remains.

Site Geology

- Exposures of bedrock present in an erosional cut near the right side of the dam allowing inspection of bedrock that would have been below the dam.
- Generally alternating horizontal beds of weak friable sandstone and mudstone (Upper Dawson Formation) below the dam’s crest overlain by stronger horizontally bedded conglomerate (Castle Rock Conglomerate) exposed in cliffs upslope from the dam.
- Three distinct mudstone units (MS1, MS2, MS3) and three distinct sandstone units (SS1, SS2, SS3) observed.
- Surficial deposits of colluvium of unknown thickness also observed.
- Colluvium appears to be more extensive in depth and aerial extent along the left portion of the dam; large boulders present in left groin area.

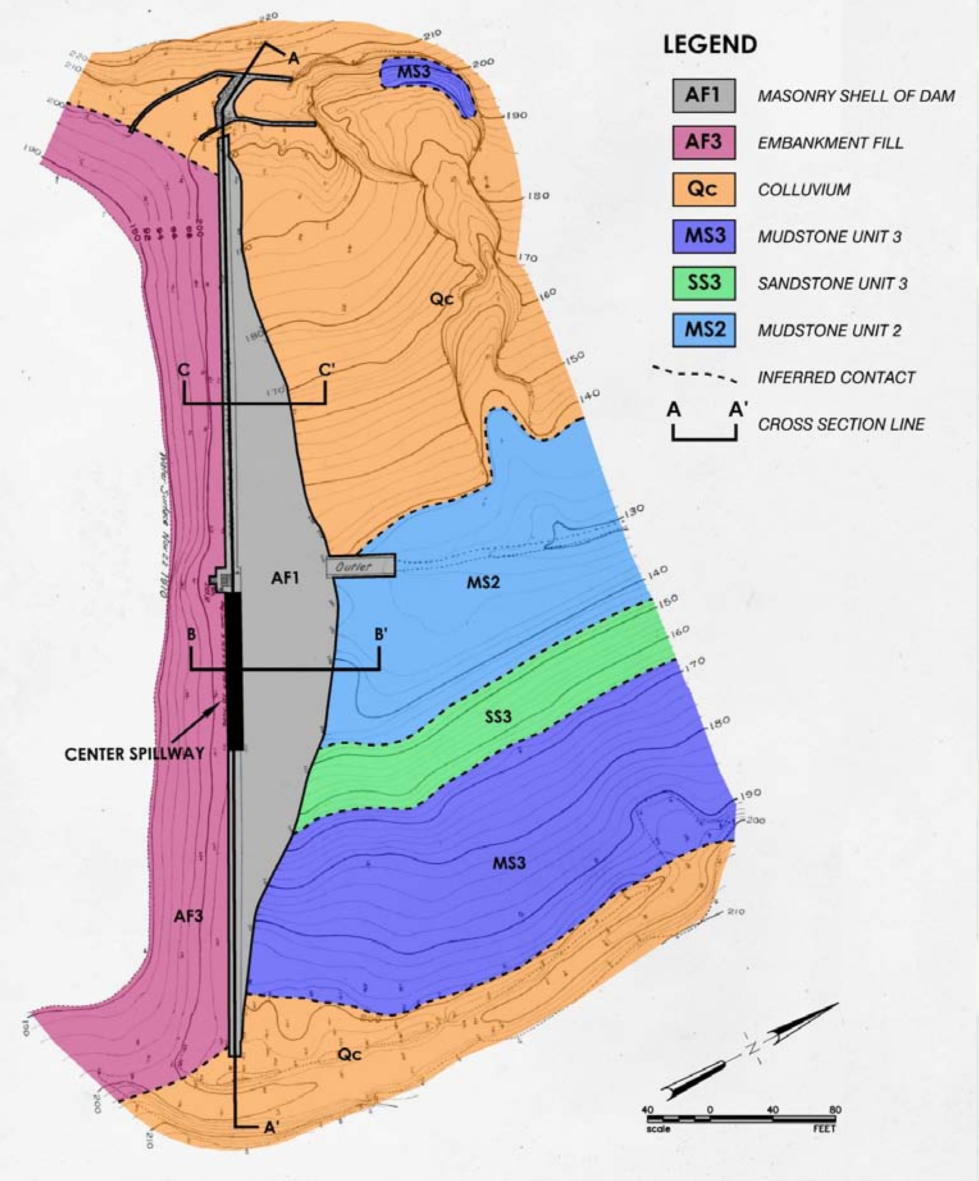
Erosional Cut on Right Side of Cherry Creek

Exposed face stands nearly vertical indicating some strength.

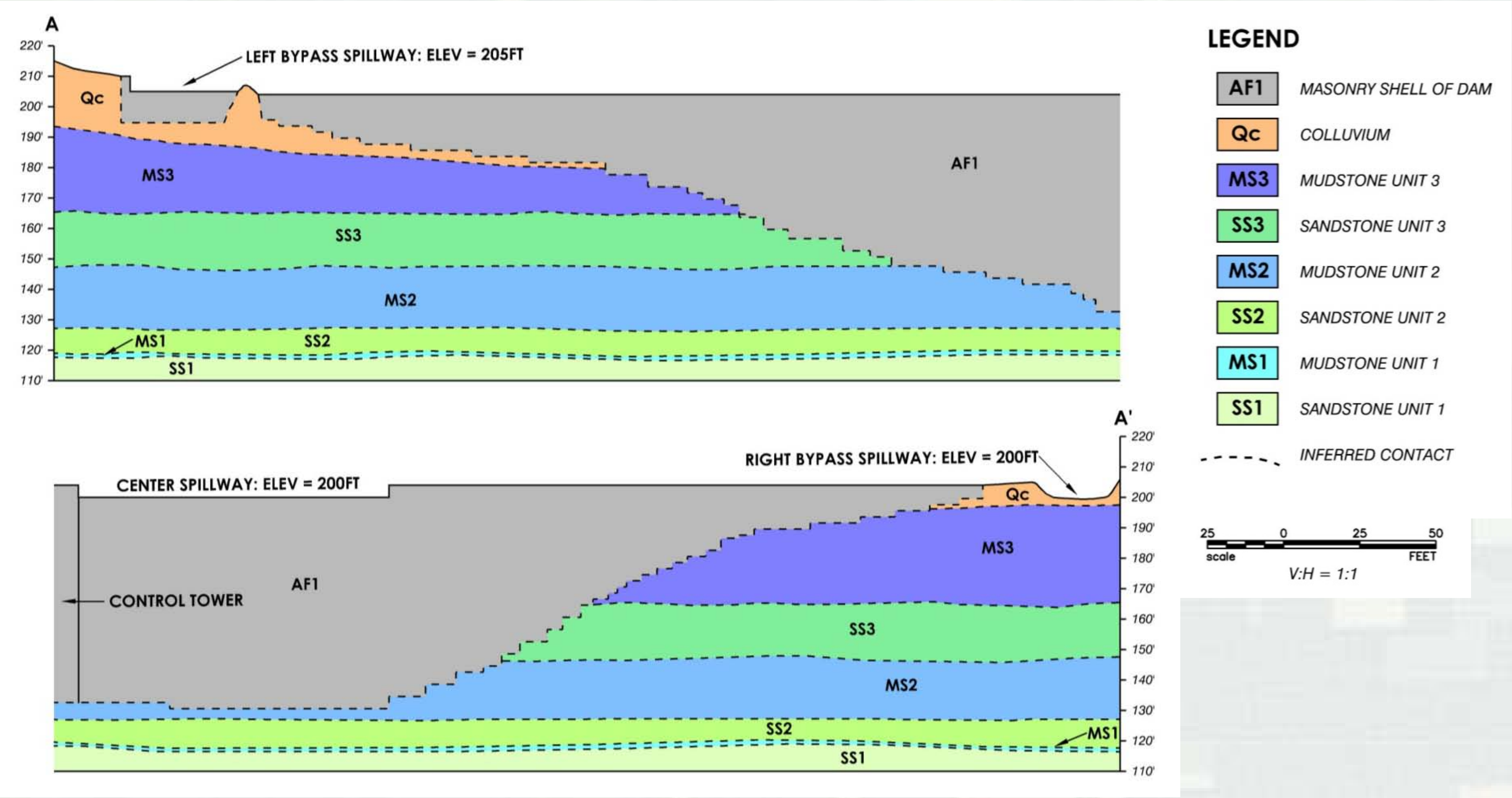


Base of dam founded on MS2, significant stream erosion subsequent to failure.

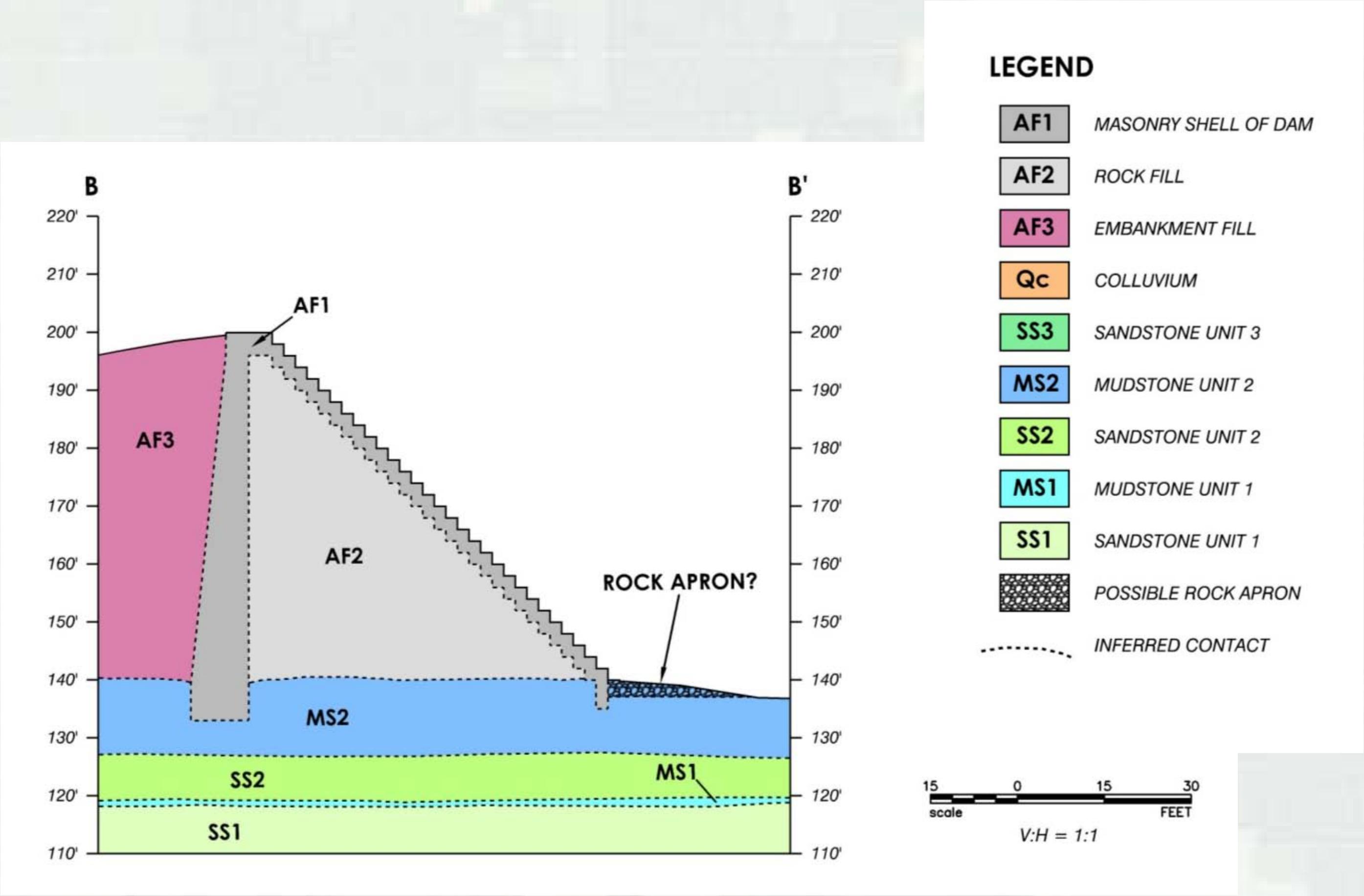
Pre-Failure Surficial Geologic Map



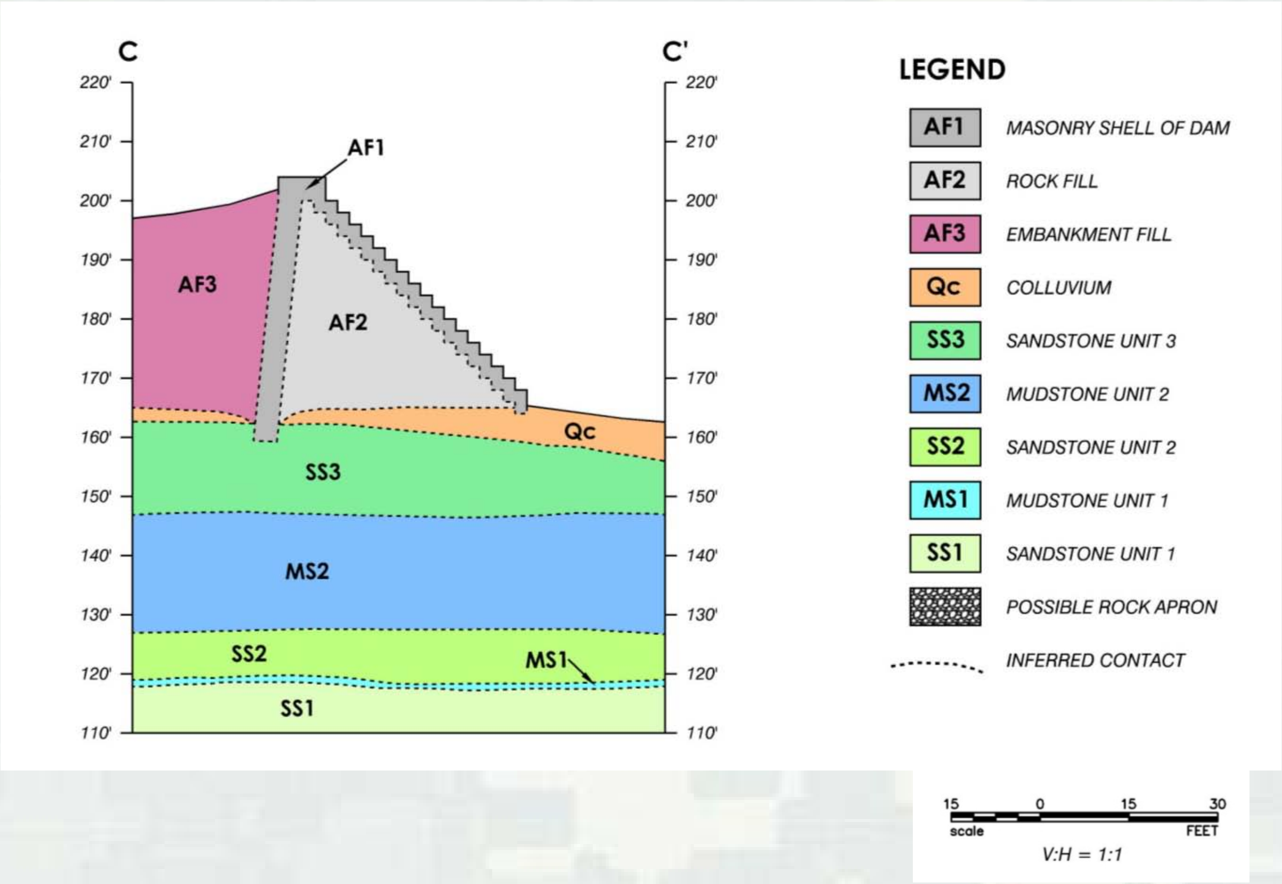
Cross Section A-A'



Cross Section B-B’



Cross Section C-C’



Site Material Descriptions

- Masonry Shell of the Dam (AF1)
  - Conglomerate blocks cemented with mortar.
  - Blocks quarried from Castle Rock Conglomerate outcrops above the dam site.
  - Lithology same as typical Castle Rock Conglomerate but with a lower percentage of cobbles and boulders.
  - Schmidt hammer UCS 14 MPa to 47 MPa average 30 MPa (4350 psi).
  - Testing on five core samples from Reuter-Hess drilling UCS 7.4 MPa to 30 MPa average 14 MPa (2030 psi).

Masonry Shell of the Dam (AF1)

Site Material Descriptions

- Rubble Fill (AF2)
  - Irregular and variable sized blocks of Castle Rock Conglomerate.
  - Some sand, silt, and clay.
  - Many large voids.
  - Scattered wood fragments (may have deposited later) including one 4 inch diameter log.

Site Material Descriptions

- Upstream Embankment Fill (AF3)
  - Dark brown clayey sand with gravel.
  - Medium dense.
  - Slightly moist.
  - Scattered cobbles, boulders, and organics.
  - Properties somewhat variable across the site.

Site Material Descriptions

- Quaternary Colluvium (Qc)
  - Orange-brown clayey gravel with sand, cobbles, and boulders.
  - Medium dense to dense.
  - Moist.
  - 50-70% gravel, cobbles, and boulders up to 6 ft diameter.
  - Scattered boulders up to 30 ft in diameter (mostly on left side of dam).
  - 1 to over 10 ft thick.

Site Material Descriptions

- Mudstone Unit 3 (MS3)
  - Dark bluish gray siltstone with trace fine sand.
  - Weak to moderately strong.
  - Fresh to slightly weathered.
  - Closely to very closely spaced tight fractures.
  - Fracture walls slightly rough to rough and planar with some iron staining.
  - Schmidt hammer results below range of instrument (<10 MPa).
  - Point load testing UCS of 1.0 to 2.6 MPa average of 1.7 Mpa (250 psi).
  - Reuter-Hess correlations UCS 0.048 to 1.5 Mpa (220 psi); RQD generally below 50% but ranging 0 to 100%.

Mudstone Unit 3 (MS3)

Site Material Descriptions

- Sandstone Unit 3 (SS3) and Sandstone Unit 2 (SS2)
  - Friable, weak, pinkish-tan, fine to coarse grained sandstone.
  - Scattered gravel up to 1 inch diameter.
  - Fresh to slightly weathered.
  - Thin to thickly bedded with cross-bedding.
  - No jointing or fracturing apparent.
  - Too weak for Schmidt hammer or point load testing.
  - Reuter-Hess one UCS test at 0.028 MPa (5 psi)
    RQD generally below 50% but ranging 0 to 100%;
    SPT testing 47 to over 200 blows per foot.

Sandstone Unit 2 (SS2)

Site Material Descriptions

- Mudstone Unit 2 (MS2)
  - Dark olive brown sandy siltstone.
  - Very weak to weak.
  - Moderately weathered.
  - Very closely spaced tight fractures.
  - Fracture walls slightly rough to rough and planar with some iron staining.
  - Schmidt hammer below range of instrument (UCS<10 Mpa – 1450 psi).
  - Point load below seating load of instrument (UCS<0.7 Mpa – 100 psi).
  - Reuter-Hess correlations same as for MS3

Mudstone Unit 2 (MS2)

Site Material Descriptions

- Sandstone Unit 1 (SS1)
  - Friable, weak, light-tan, fine grained sandstone.
  - Fresh to slightly weathered.
  - Thinly bedded.
  - No jointing or fracturing apparent.
  - Too weak for Schmidt hammer or point load testing.
  - Material properties more like soil than rock.
  - USCS would classify this as very weakly cemented, medium dense, poorly graded sand.
  - Reuter-Hess correlations show core recovery and RQD near 0% for similar material.

What are the Vulnerabilities or Signs of Distress that Would Lead Us to a Potential Failure Mode?

Potential Vulnerabilities

- Leakage through left side of masonry and bulge in this location – masonry movement, loss of support
- Rock weak and deformable – excessive deformation of masonry
- Rock erodible – spillway flow erosion and undermining
- Spillway capacity unknown, what level of flood can be passed?
- Rubble masonry not grouted, drainage behind grouted faces unknown, stability of wall?

Castlewood Dam Failure

- Failure occurred at ~12:00AM August 3, 1933 following a heavy rainfall.
- Rainfall in upper drainage basin of Cherry Creek estimated at 4 to 9 inches over 3 hours.
- Only witness to failure was the dam tender (Hugh Paine).

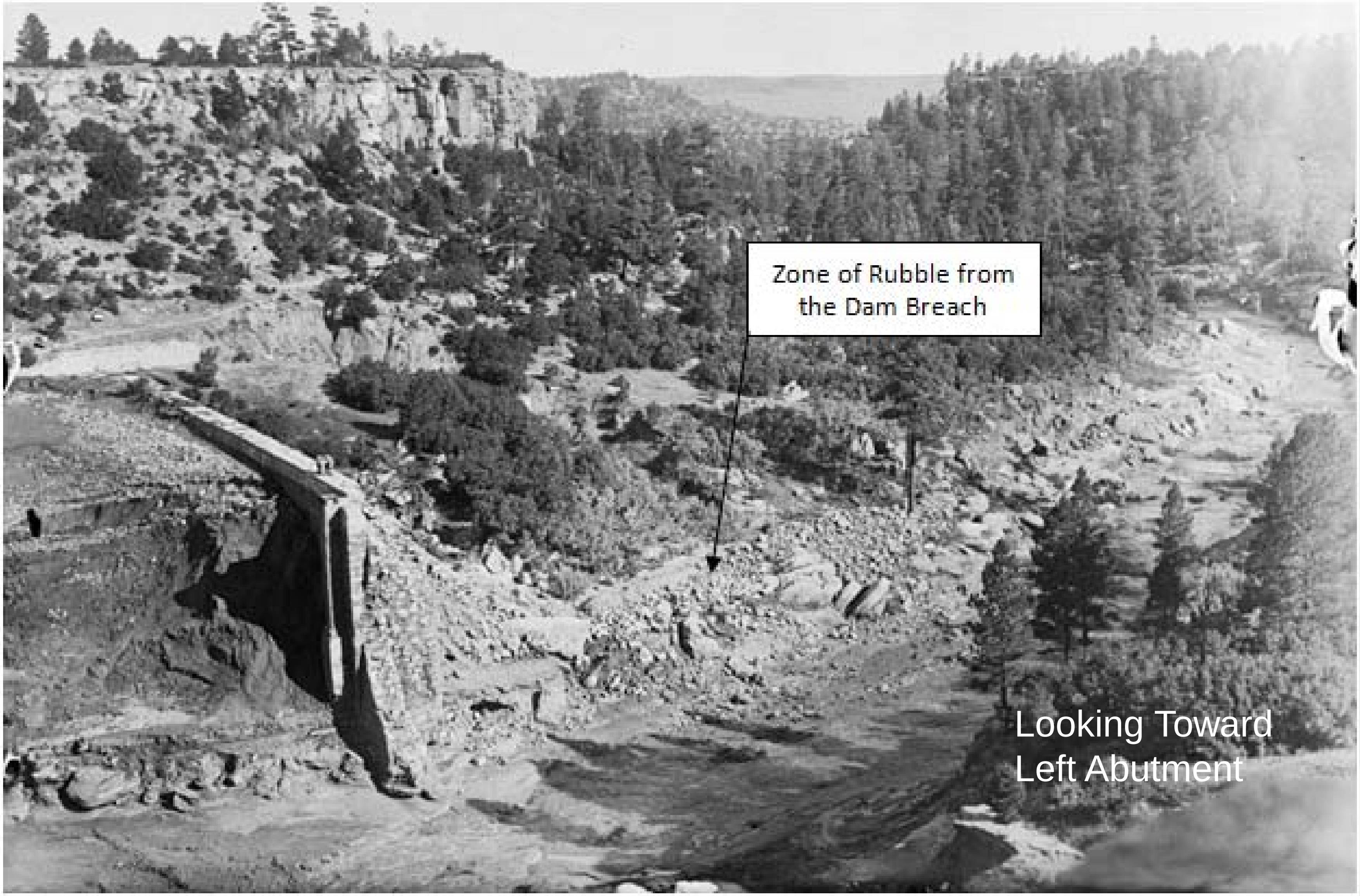
Castlewood Dam Failure

- The following statements were made by the dam tender concerning events leading up to the failure.
  - 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 (~1 ft over dam crest) and through the dam, and within a few minutes the surface of the reservoir had dropped thirteen feet below the spillway”.
  - “The lapse of time between the inrush of water into the reservoir and the time of the dam failure did not exceed 45 minutes”

Castlewood Dam Failure

- After witnessing the dam failing, the dam tender left the site to warn people.
- 15 ft high flood wave reached Denver at 6:20 AM.
- Flood wave caused $1,000,000 in damage (1933 dollars) and two deaths.
- Low loss of life attributed to advanced warning for evacuation.
- A few low quality photos discovered showing dam after breach.
- Extent of erosion not completely evident.

Photo Shortly after Dam Breach

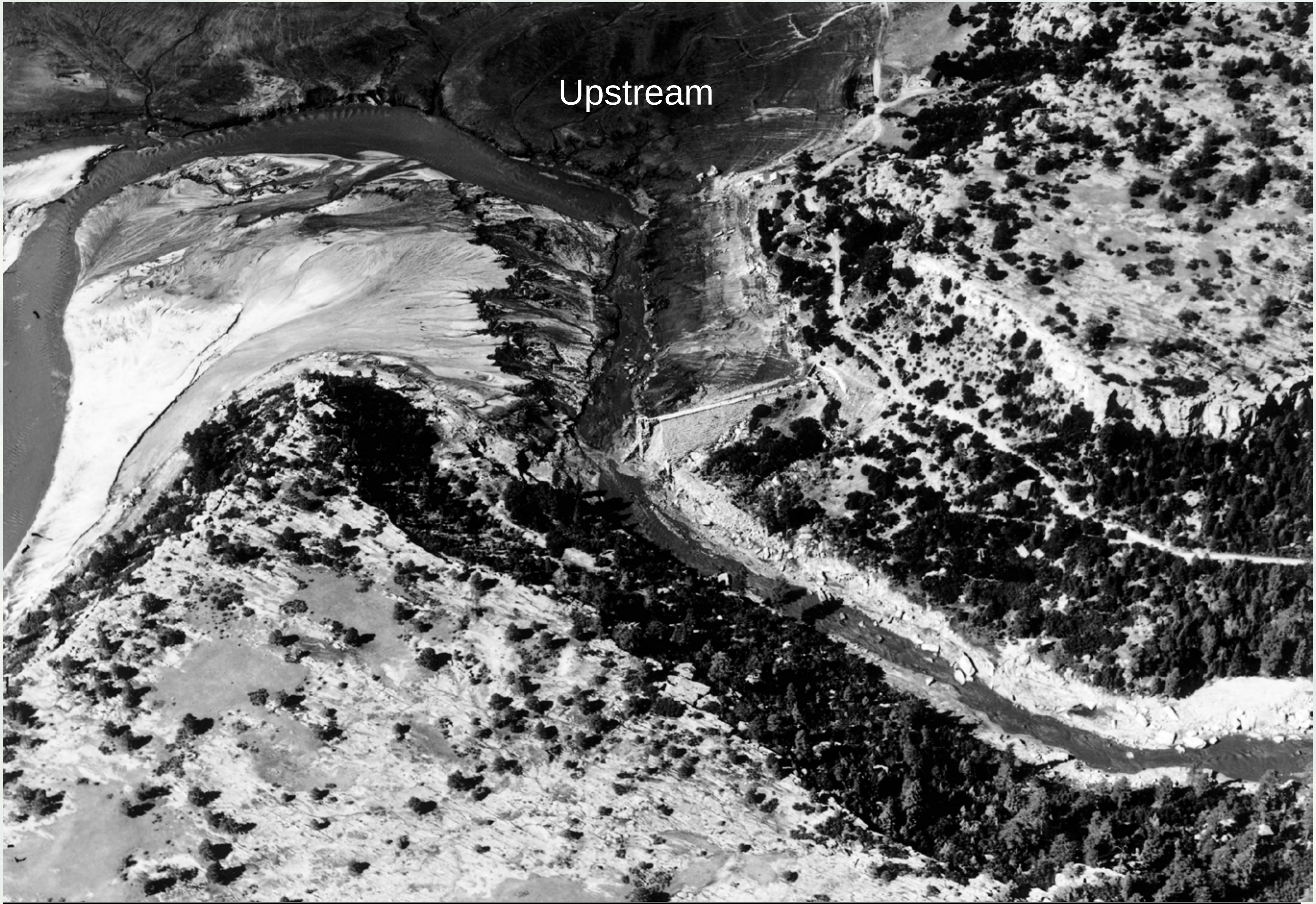


(Photo courtesy of Denver Public Library)

Photo Shortly after Dam Breach

(Photo courtesy of Denver Public Library)

Photo Shortly after Dam Breach



(Photo courtesy of Denver Public Library)

Close-up of Last Photo

(Photo courtesy of Denver Public Library)

Castlewood Dam Failure

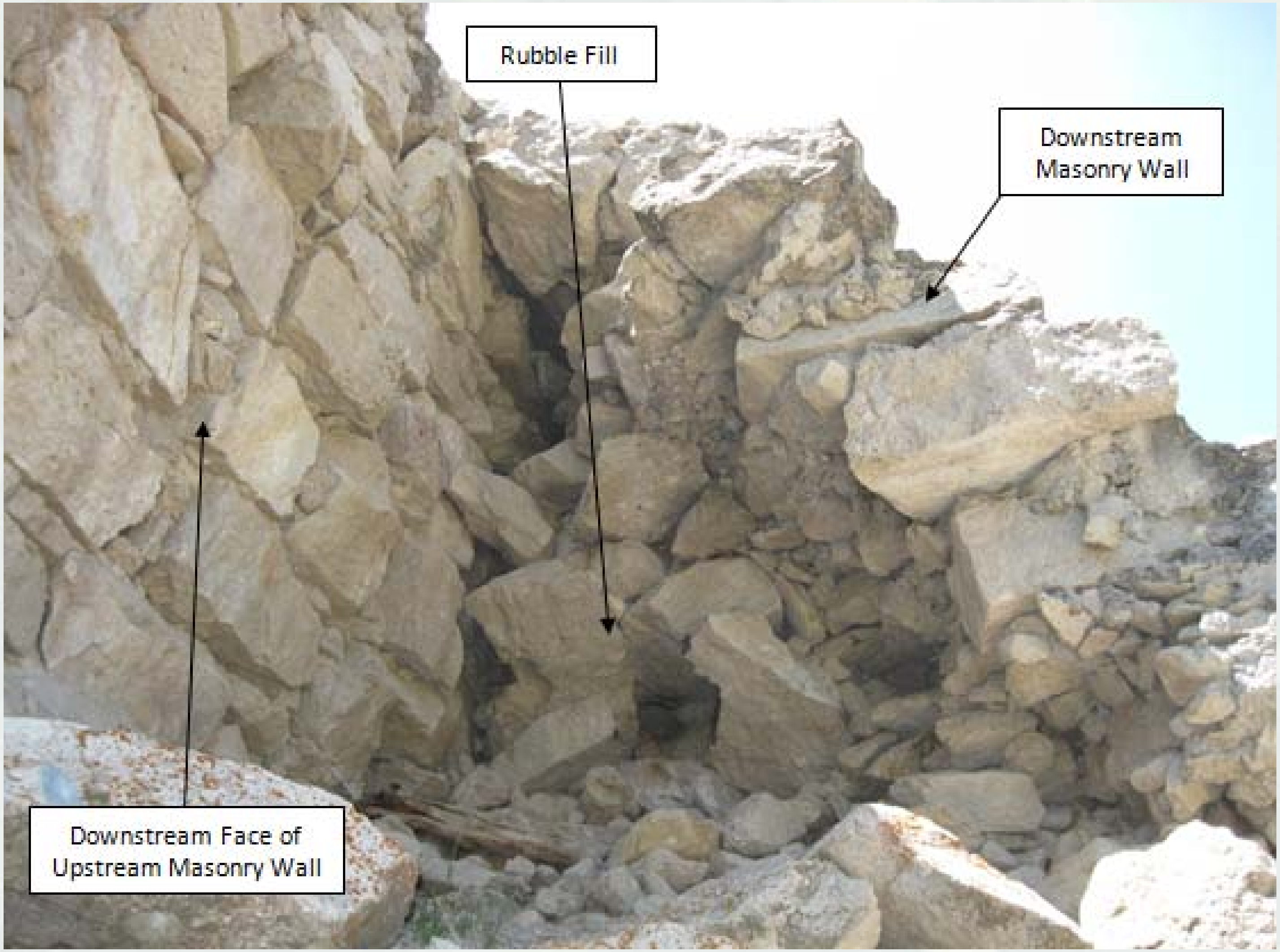
- Reservoir level was 53.34 ft (1.34 ft above dam crest) according to the State Engineer's report.
- Maximum discharge of 126,000 cfs estimated during failure.
- Maximum flow rate measured in Cheery Creek in Denver was 16,000 cfs.

Current Conditions – Right Side

Current Conditions – Right Side

Upstream Side of Left Portion of Remains

Exposed Portion of Dam Looking Towards Left Abutment



Foundation of Upstream Masonry Wall

Crest of Dam looking towards Right Abutment

Remains of Upstream Embankment

Left Bypass Spillway

Left Bypass Spillway – Looking Downstream

Audience Participation

- What are the potential failure modes?
- What analysis could be done to support/refute these potential failure modes?

Official State Engineer Investigation

- Post failure investigation by the Colorado State Engineer concluded that failure was due to erosion at the lower toe of the dam <u>or</u> plucking of blocks from the downstream masonry wall.
- No other investigations of the failure took place.

Scour Analysis

- Scour analysis using the Erodibility Index Method (Annandale, 1995) was utilized under the guidance and review of Dr. George Annandale.
- Other methods explored but since the hydraulics were complex, the site materials were highly variable, and sampling was not possible, Erodibility Index Method was chosen.
- Method is generally performed by quantifying the erosive resistance of the materials subject to scour and erosive capacity of the water.
- Comparisons of these values are then conducted to assess whether scour would be predicted.

Scour Analysis Methods

- Erosion Resistance of Site Materials

$$K = M_s \cdot K_b \cdot K_d \cdot J_s$$

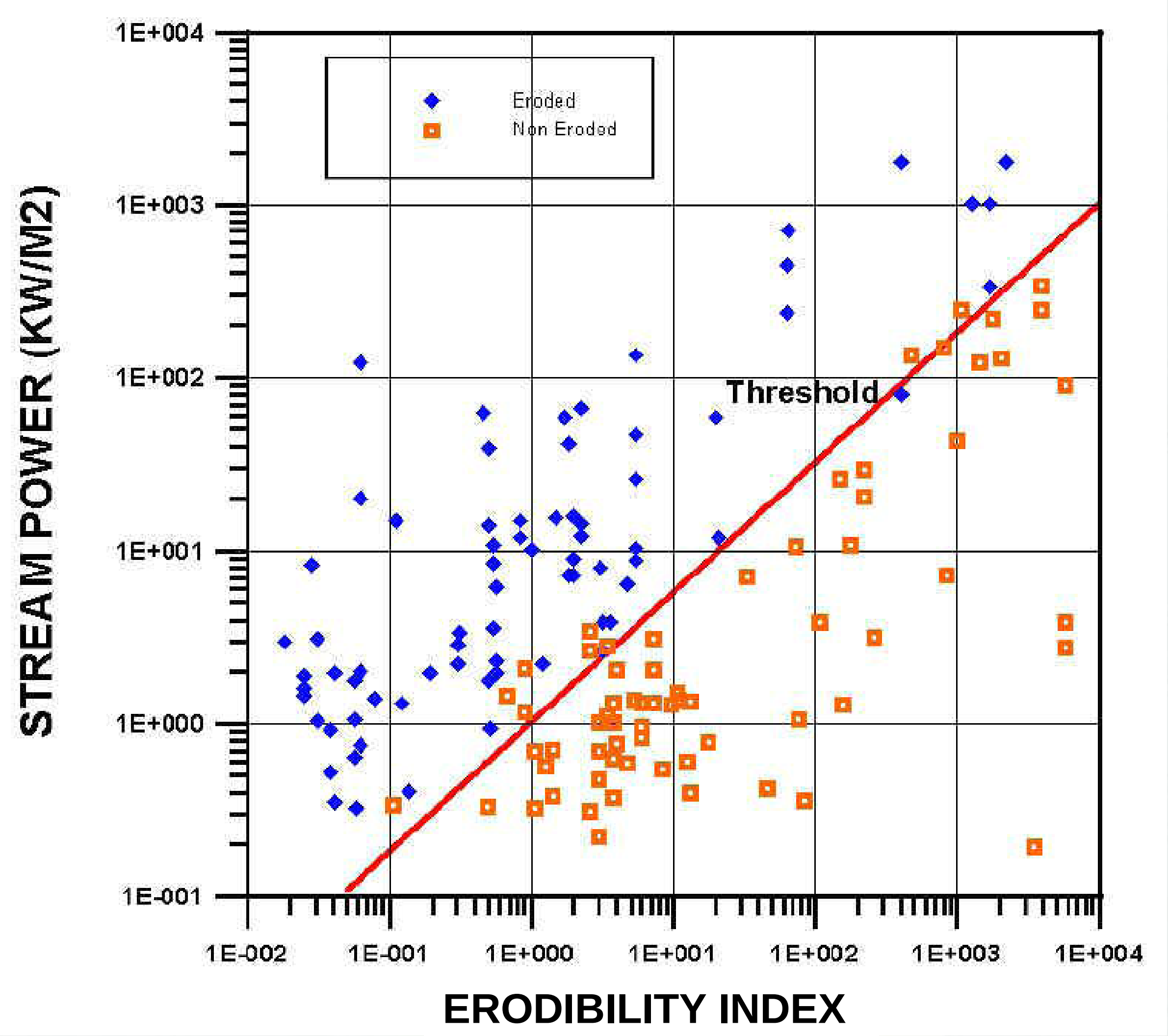
Where: K = erodibility index.
M_s = mass strength number of material.
K_b = block size number = RQD/(joint set number (J_n)) for rock, $1000 \cdot$(characteristic particle diameter in meters (D))^3 for non-cohesive soils, and 1.0 for cohesive soils such as cemented sand.
K_d = inter-block shear strength number = (joint roughness number (J_r))/(joint alteration number (J_a)) for rock or tan(internal angle of friction (ϕ)) for soils.
J_s = shape and orientation number.

  - Values chosen using tables in Annandale (2006).
  - High and low end estimates of the erodibility index were calculated for each material.

Scour Analysis Methods

- Stream Power represents erosion potential of flowing water.
- Extension of “power” needed to rip rock by bulldozer.
- Defined as rate of energy dissipation, kW/m^2.

Threshold Line



Scour Analysis Methods

- Erosion threshold stream power – stream power at which erosion would just begin to occur.

$$P_c = K^{0.75} \; if \; K > 0.1$$
$$P_c = 0.48 \cdot K^{0.44} \; if \; K \leq 0.1$$

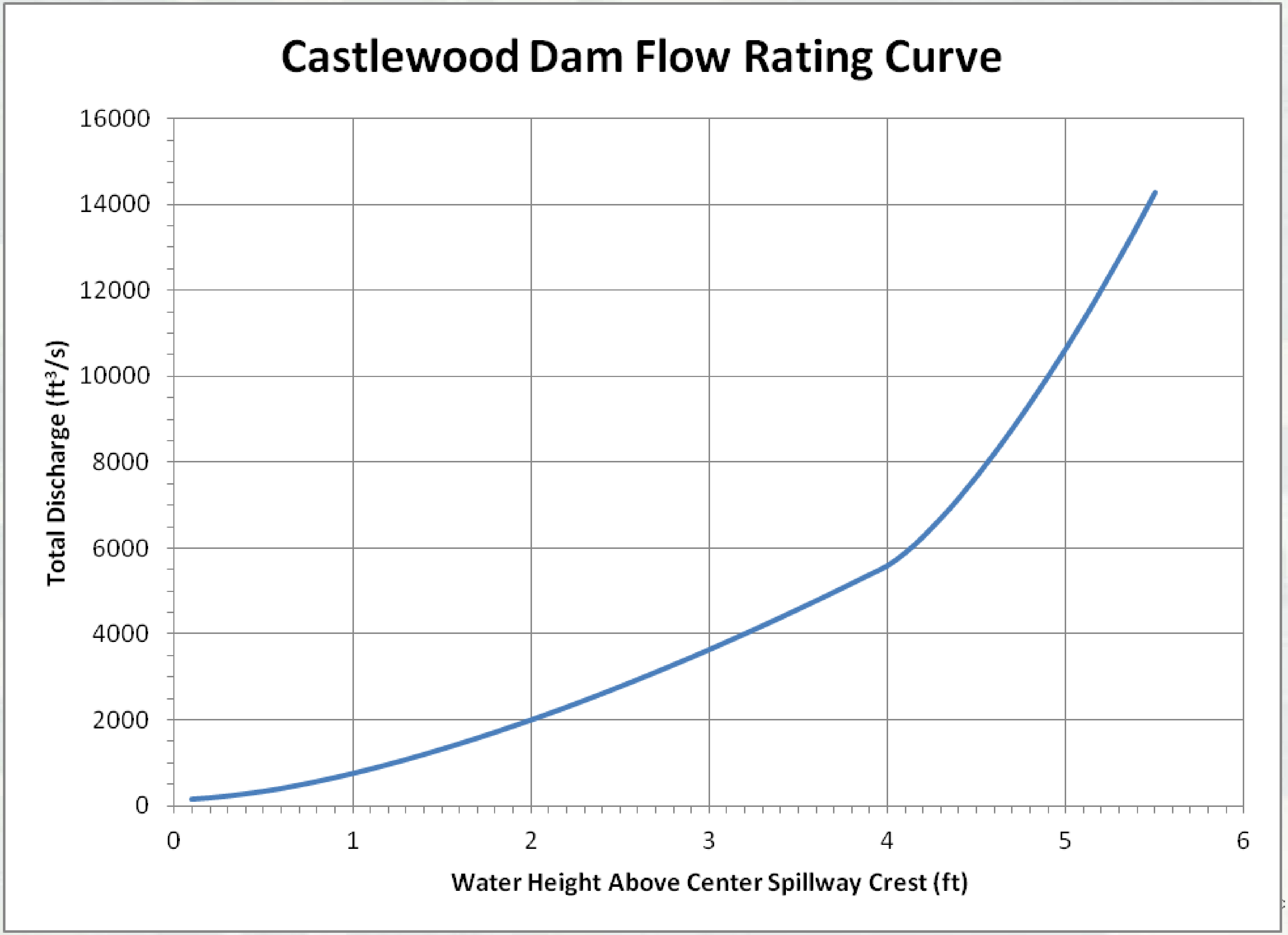
Where: P_c = erosion threshold stream power (kW/m^2).
K = erodibility index.

Scour Analysis Methods

- Flow rating curve first had to be established to quantify the erosive capacity of the water.
- Conservative estimates of flow rate quantified using the equation for critical depth of flow through a rectangular channel.

$$Q = b\sqrt{g \cdot y_c^3}$$

Where: Q = flow rate (m^3/s).
b = flow width (m).
g = acceleration of gravity (m/s^2).
y_c = depth of flow over spillway (m) (assumed to be at critical flow depth).



Castlewood Dam Flow Rating Curve
16000
14000
12000
10000
8000
6000
4000
2000
0
0
1
2
3
4
5
6
Total Discharge (ft³/s)
Water Height Above Center Spillway Crest (ft)

Scour Analysis Methods

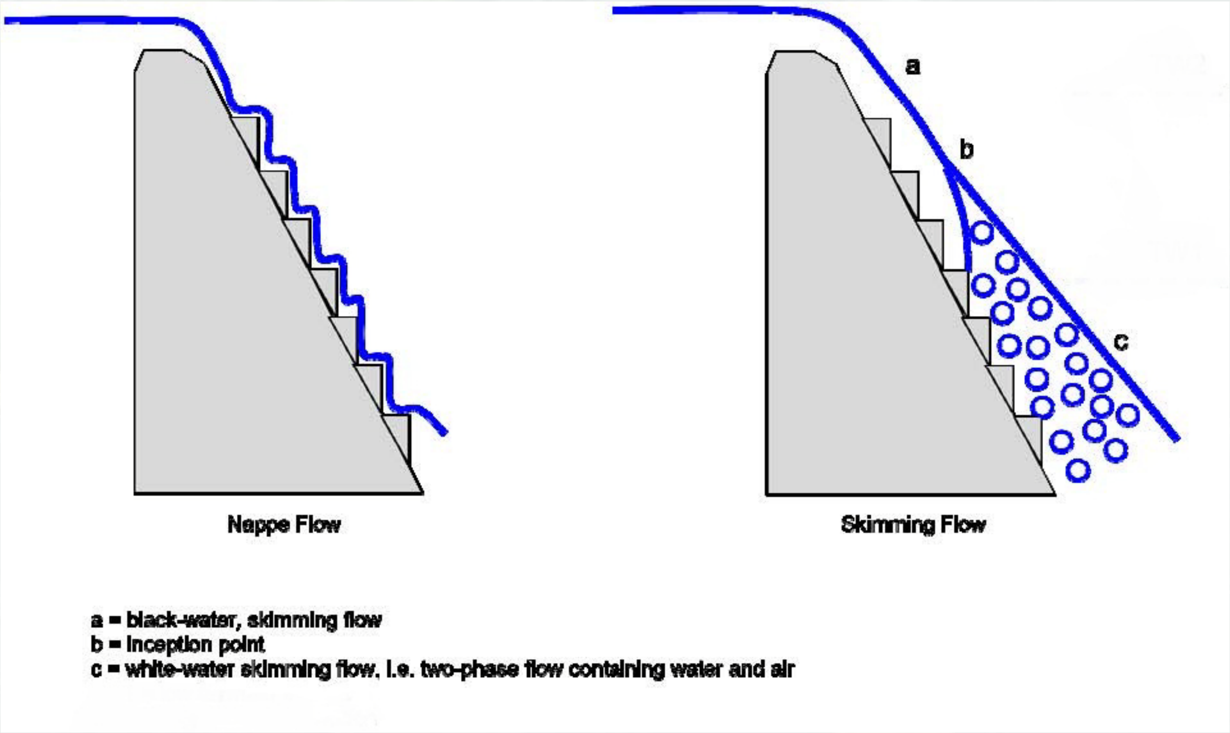
- Tailwater vs No-Tailwater
  - Based on accounts of overtopping event resulting in failure, the reservoir level rose quickly and it is unlikely that steady state tailwater conditions developed.
  - It is likely that for many of the previous center spillway flow events that the reservoir level also rose quickly due to heavy rainfall and that steady state tailwater conditions did not occur for many of these either.
  - To assess the complete range of potential conditions, HEC-RAS was used to model expected tailwater with steady state flow conditions and stream power at the toe of the dam was quantified for both no tailwater and steady state tailwater conditions.

Tailwater Depth vs. Height Above Center Spillway Crest for Steady State Tailwater Scenario

Water Height Above Center Spillway Crest (ft)	Total Discharge (cfs)	Tailwater Depth (ft)
0.5	324	0.0
1.0	741	0.6
1.5	1307	2.1
2.0	1988	3.5
2.5	2767	5.0
3.0	3632	6.4
3.5	4575	7.8
4.0	5590	9.2
4.5	7675	11.6
5.0	10006	14.5
5.34	13066	16.5

Scour Analysis Methods

- Nappe Flow and Skimming Flow Transition



Scour Analysis Methods

- Critical Depth of Flow for Transition from Nappe to Skimming Flow

$$h_c = S \cdot (0.91 - 0.14 \tan(\varphi))$$

Where: h_c = critical depth of flow over spillway at which skimming flow initiates (m).
S = step height (m).
φ = angle of the downstream dam face.

  - Since this study was concerned with high flow events that likely resulted in scour, and steps would likely dissipate energy for low flow events, flow depths below h_c were not analyzed further.

Scour Analysis Methods

- Stream Power at the Toe of the Dam or Tailwater Elevation
  - Quantified for various flow depths using methodologies presented by Boes and Hager (2003).

- Further calculations completed to account for energy dissipation of flow as it plunges below the tailwater surface
  - Energy dissipation calculations performed according to Annandale (2006).
  - Calculate stream power at base of potential scour hole.

Scour Analysis Methods

- Stream Power along Groins
  - Rough estimates of stream power along the left and right groin during overtopping were also calculated.
  - Equation for stream power expended at the bed surface in open channel flow used (reasonable range of flow widths assumed).

$$SP = \gamma \cdot \frac{Q}{W} \cdot S_f$$

Where:
SP= stream power at base of flow (kW/m^2).
γ= unit weight of water (kN/m^3).
Q= total flow rate at specified point along groin (m^3/s).
W= estimated width of flow (m).
S_f= energy slope (assumed to be average slope of groin).

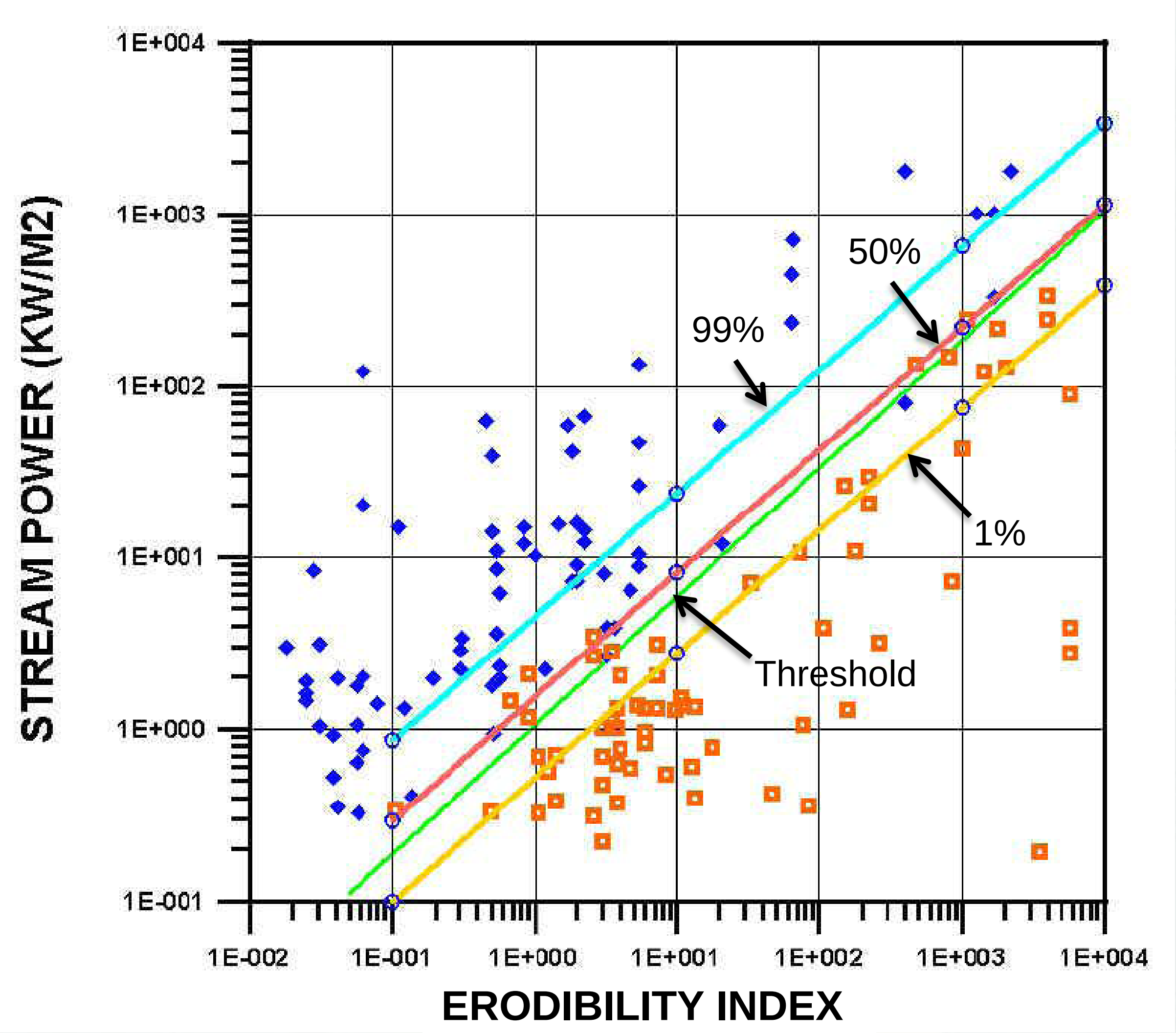
Scour Analysis Methods

- Prediction of Scour Extent
  - ► Simplest way is to directly compare the stream power of the water to the erosion threshold stream power of the material.
    - If stream power is greater than erosion threshold stream power then scour is predicted and vice-versa.
  - ► Also can use logistic regression equations developed by Wibowo, et. al. (2005) to give a probability of scour.

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

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

Wibowo Analysis



Scour Analysis Results

- **Erosion Resistance of Site Materials**
  - SS2 and SS3 showed properties borderline between rock and soil so they were characterized as both.
  - SS1 was considered too weak to be considered as a rock so it was only characterized as soil.
  - All mudstones and the masonry shell were characterized as rock.
  - Colluvium was characterized as a soil.

Erosion Resistance of Rock

Material		UCS (MPa)	M_s	RQD	J_n	K_b	J_r	J_a	K_d	J_s	K	P_c (kW/m^2)
AF1	High	47	35.0	100	2.73	36.6	4.0	1.0	4.0	1.20	6150	695
	Low	14	17.7	90	2.73	33.0	1.5	1.0	1.5	0.54	473	101
MS3	High	3.3	1.86	100	1.00	100	3.0	1.0	3.0	0.51	285	69.3
	Low	1.7	1.86	5	2.73	1.83	1.0	1.0	1.0	0.51	1.74	1.51
SS3	High	1.7	0.87	100	1.00	100	4.0	1.0	4.0	1.00	348	80.6
	Low	0	0.87	5	1.00	5.00	4.0	1.0	4.0	1.00	17.4	8.52
MS2	High	<1.7	0.87	100	1.00	100	3.0	1.0	3.0	0.51	133	39.2
	Low	<1.7	0.87	5	2.73	1.83	1.0	1.0	1.0	0.51	0.81	0.86
SS2	High	1.7	0.87	100	1.00	100	4.0	1.0	4.0	1.00	348	80.6
	Low	0	0.87	5	1.00	5.00	4.0	1.0	4.0	1.00	17.4	8.52
MS1	High	<1.7	0.87	100	2.73	36.6	3.0	1.0	3.0	0.51	48.8	18.5
	Low	<1.7	0.87	5	2.73	1.83	1.0	1.0	1.0	0.51	0.81	0.86

Erosion Resistance of Soil

Material		Density	M_s	Characteristic Grain Size (m)	K_b	ϕ	K_d	J_s	K	P_c (kW/m²)
Qc	High	Very Dense	0.41	1.0	1000	50	1.19	1	489	104
	Low	Dense	0.19	0.152	3.51	36	0.73	1	0.48	0.58
SS3	High	Very Dense	0.41	6.35E-03	1*	50	1.19	1	0.49	0.35
	Low	Dense	0.19	6.35E-03	1*	36	0.73	1	0.14	0.20
SS2	High	Very Dense	0.41	6.35E-03	1*	50	1.19	1	0.49	0.35
	Low	Dense	0.19	6.35E-03	1*	36	0.73	1	0.14	0.20
SS1	High	Loose	0.09	1.25E-04	1*	36	0.73	1	0.07	0.14
	Low	Medium Dense	0.04	1.25E-04	1*	29	0.55	1	0.02	0.09

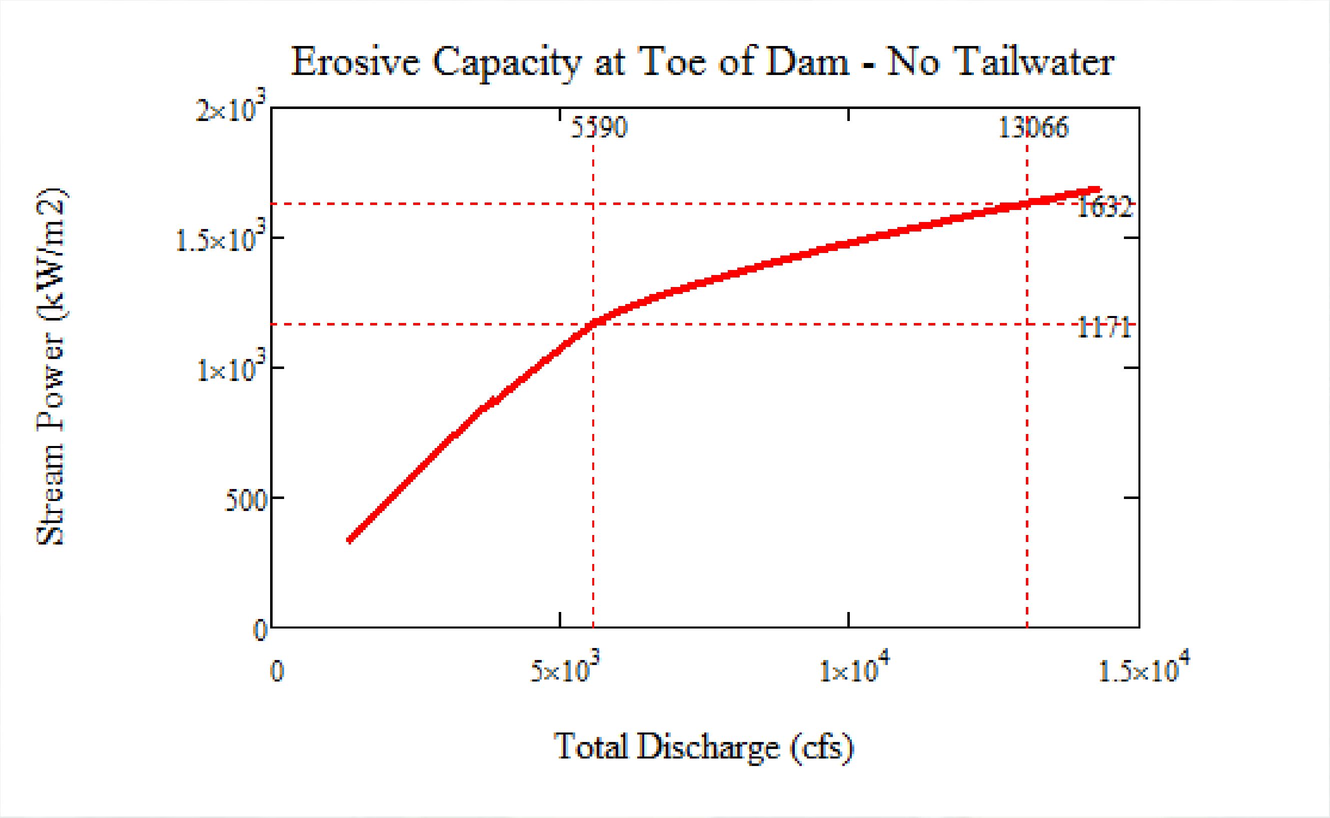
Scour Analysis Results

- **Assessment of Flow Conditions**
  - Calculations showed that skimming flow would occur at critical flow depths over 1.53 ft (equivalent to 1346 cfs).
  - Further analysis limited to flows ranging from 1346 cfs to 13,066 cfs (maximum flow event during failure).

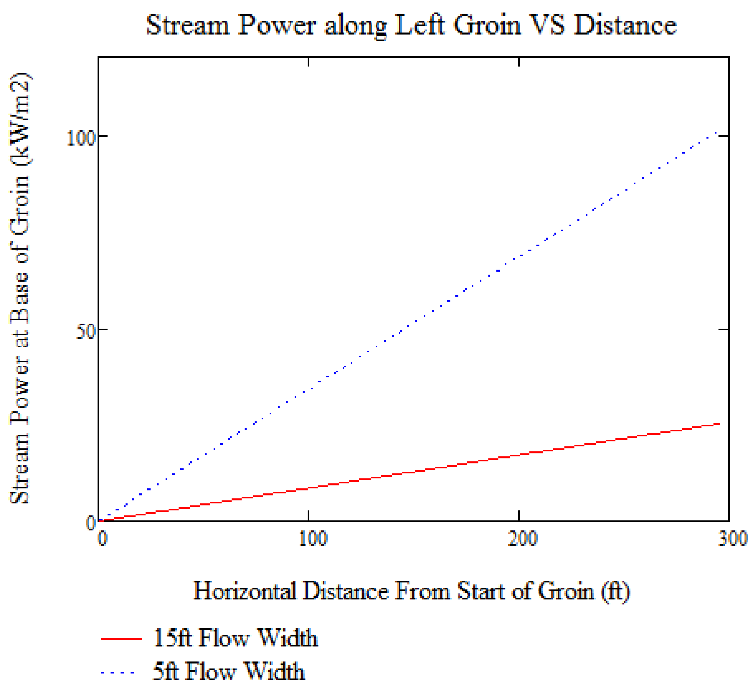
Key Reservoir Elevations

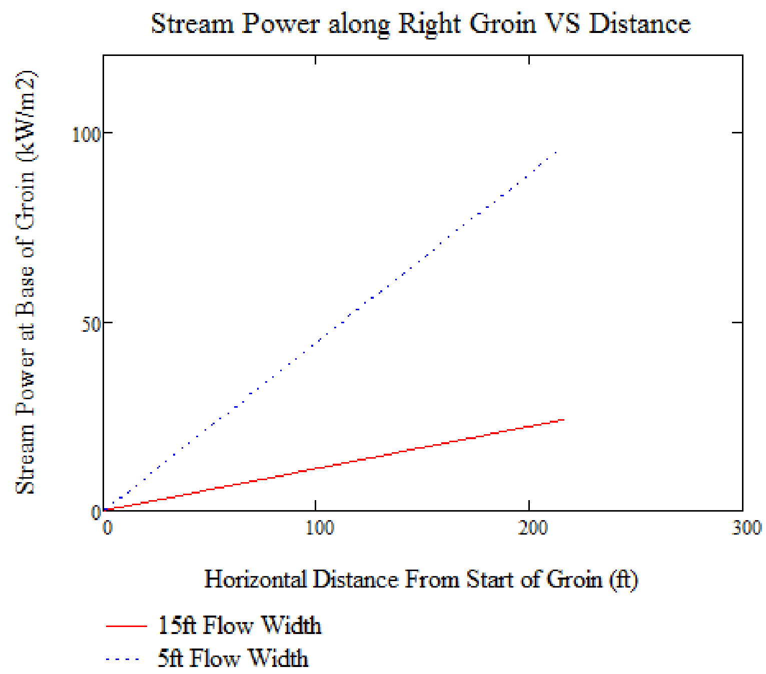
Height above center spillway crest	Corresponds to
1.53 ft	Threshold for skimming flow
2 ft	1902-1933 max level achieved several times
4 ft	1924 flood event resulted in 4 ft deep scour hole
5.34 ft	Estimated depth at time of failure - Aug 3, 1933

Erosive Capacity of Water at Toe of Dam Assuming no Tailwater



Erosive Capacity of Water Along Groins at 5.34 ft of Overtopping Assuming no Tailwater



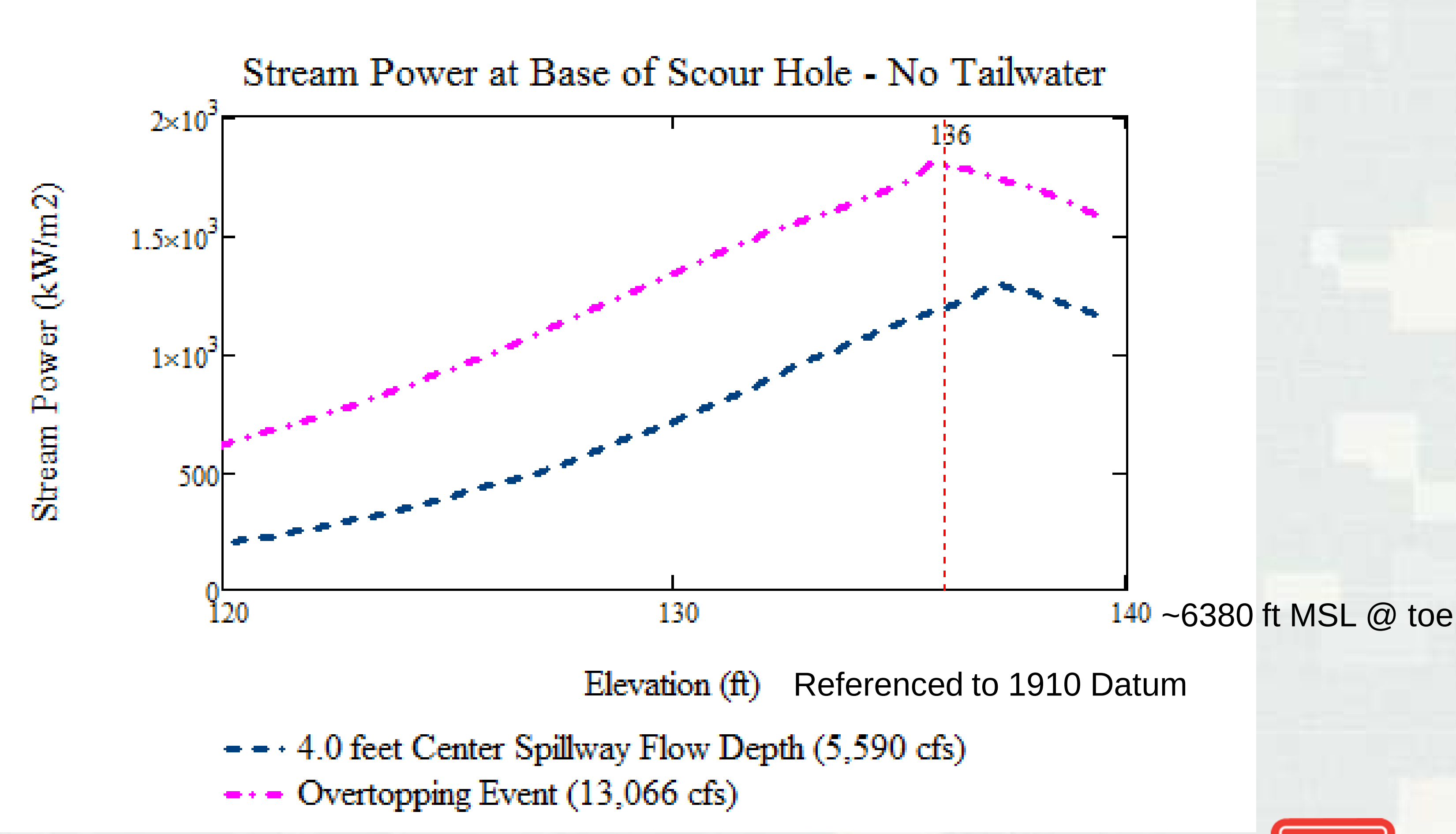


Varies greatly with width of flow
Added to stream power at toe of center spillway

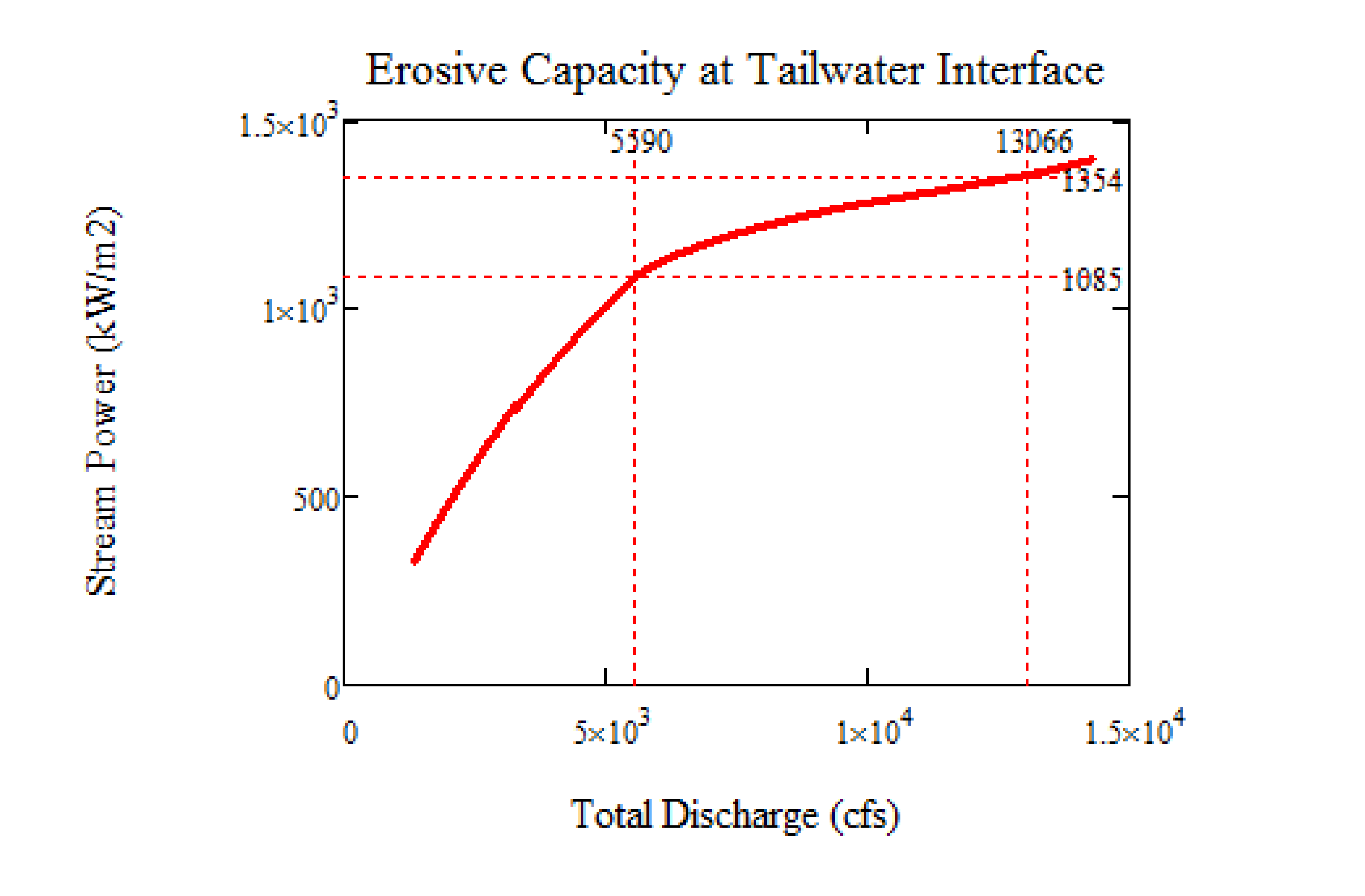
Stream Power at Toe of Dam – No Tailwater

Center Spillway Flow Depth (ft)	Total Discharge (cfs)	Estimated Stream Power at Toe (kW/m^2)
1.53	1346	330
2.00	1988	482
4.00	5590	1171
5.34	13066	1681 to 1830

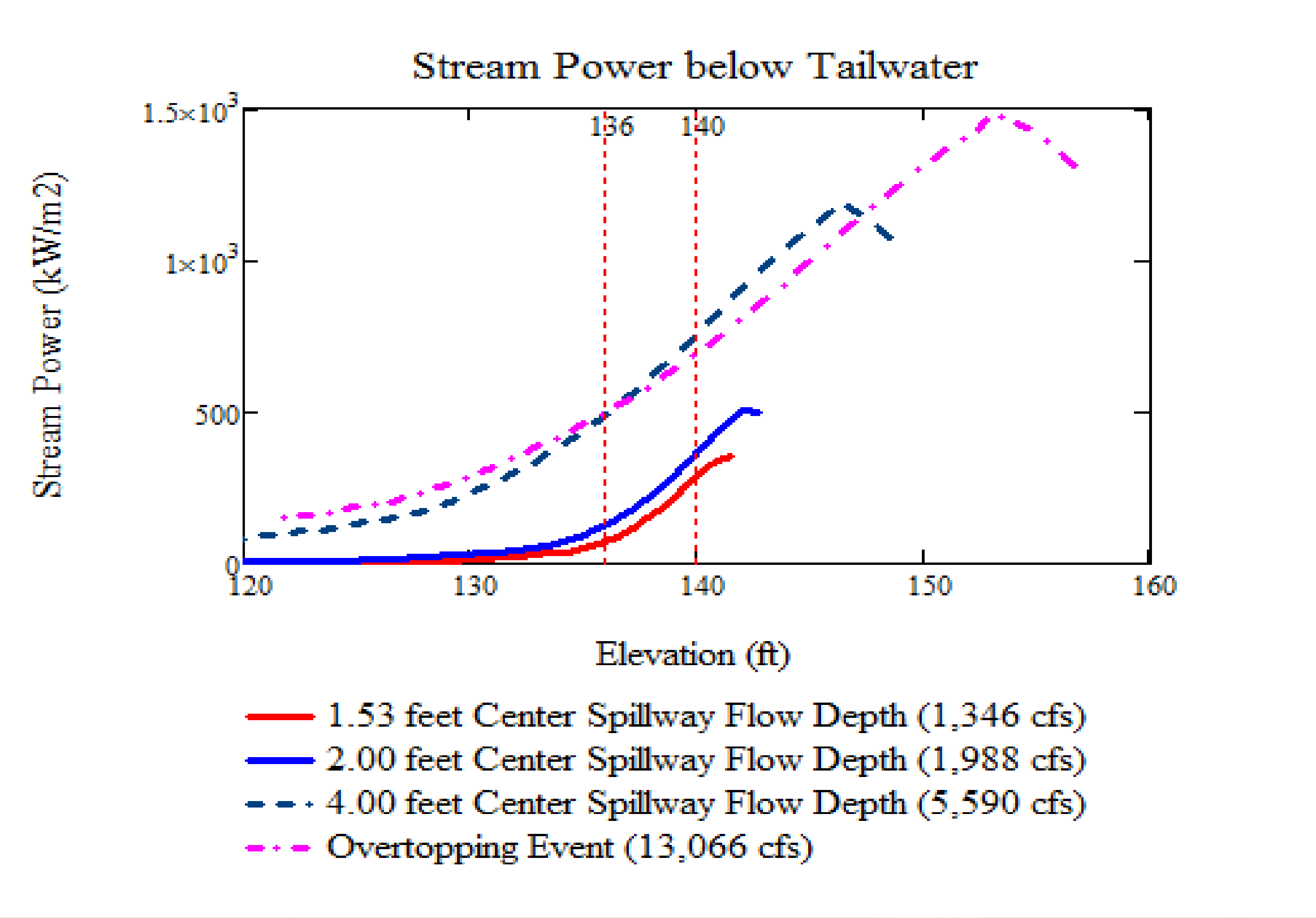
Stream Power vs. Base of Scour Hole Elevation assuming no Tailwater



Erosive Capacity of Water at Toe of Dam Assuming Steady State Tailwater Conditions



Stream Power Below Tailwater Interface vs. Elevation



Stream Power at Toe of Dam – Steady StateTailwater

Center Spillway Flow Depth (ft)	Total Discharge (cfs)	Estimated Stream Power at Toe (kW/m^2)
1.53	1346	286
2.00	1988	367
4.00	5590	756
5.34	13066	692

Prediction of Scour Extent – Toe of Dam – Unprotected Bedrock (MS2)

Center Spillway Flow Depth (ft)	Total Discharge (cfs)	No Tailwater - Estimated Stream Power at Toe (kW/m^2)	Steady State Tailwater - Estimated Stream Power at Toe (kW/m^2)	Erosion Threshold Stream Power of MS2 (kW/m2)	Scour Predicted?	Probability of Scour (Equation 7)
1.53	1346	330	286	0.86 to 39.2	Yes	> 0.99
2	1988	482	367		Yes	> 0.99
4	5590	1171	756		Yes	> 0.99
5.34	13066	1681 to 1830	692		Yes	> 0.99

Prediction of Scour Extent – Toe of Dam – Masonry Shell of Dam (AF1)

Center Spillway Flow Depth (ft)	Total Discharge (cfs)	No Tailwater - Estimated Stream Power at Toe (kW/m^2)	Steady State Tailwater - Estimated Stream Power at Toe (kW/m^2)	Erosion Threshold Stream Power of AF1 (kW/m2)	Scour Predicted?	Probability of Scour (Equation 7)
1.53	1346	330	286	101 to 695	Possible	0.01 to 0.98
2	1988	482	367		Possible	0.03 to > 0.99
4	5590	1171	756		Mostly	0.43 to > 0.99
5.34	13066	1681 to 1830	692		Mostly	0.34 to > 0.99

Note: If rock apron was present, it may have approached this material in terms of erosion resistance.

Prediction of Scour Extent – Groins

Groin Material	Horizontal Distance Along Groin to Point of Maximum Discharge for Given Material (ft)	Estimated Stream Power at Point of Maximum Discharge (kW/m^2)	Erosion Threshold Stream Power (kW/m^2)	Scour Predicted?	Probability of Scour (Equation 7)
Qc	275	24 to 94	0.58 to 104	Possible	< 0.01 to > 0.99
MS3	140	16 to 62	1.51 to 69.3	Mostly	< 0.01 to > 0.99
SS3	217	24 to 96	0.20 to 80.6	Mostly	< 0.01 to > 0.99

Empirical Evidence

- May & Aug 1897 – wave action and flow through cracks in dam erode right abutment rock and undermining portions of the dam.
- This suggests abutment rock is erodible.

Empirical Evidence

- April 1900, discharge through left spillway for 30 hours (up to 500 cfs total release both spillways) produced extensive erosion.
- Flow depth through left spillway would have been shallow (probably < 2 ft).
- Low flow depth and shallow abutment slope (~15^{o}) suggest stream power would have been small.
- Therefore, erosion resistance also small.
- Most of the erosion took place in colluvium but it appears to have extended into the underlying rock (MS3).

Empirical Evidence

- 1902 – 1933, 2 ft of flow occurred through central spillway on several occasions.
- No reported scour at of downstream face or toe of dam.
- Even at upper end of erodibility index, erosion of MS3 at toe of dam is predicted to be highly likely (max threshold streampower 69.2, actual est. 367-482 w & w/o tw P(erosion) > 99%).
- Flow may have been so short that erosion was too little to be noticed.
- Or, more likely, the rock apron was actually in place protecting the toe (threshold stream power avg. near 400, and as high as 695).

Empirical Evidence

- 1924, with 4 ft water going over central spillway, 4 ft deep scour hole developed at toe of dam.
- Rock apron 3 to 6 ft thick.
- Threshold stream power rock apron avg 400, max 695.
- Actual est. stream power 756-1171 w/ and w/o tw.
- Erosion of rock apron predicted to be highly likely, >99%.
- Erosion also predicted for downstream face near toe, but no damage reported – stream power less on slope than for apron, effect of steps and skimming flow? Downstream face likely of higher quality than rock apron.
- Although uncertainties exist, a consistent set of plausible scenarios can be formulated.

Discussion

- The fact that Castlewood Dam failed during an overtopping event does not necessarily mean that overtopping erosion was the primary cause of failure.
- Could be high loading causing excessive deformation and structural failure or development of a critical gradient.

Discussion

- Historical Evidence Supporting Overtopping Erosion Failure Mode
  - Reservoir levels show the dam was regularly subjected to high loads without evidence of distress for at least 20 years prior to failure.
  - Reservoir level during failure only 1 ft higher than the maximum previous level and 3 to 5 ft higher than spring reservoir levels that occurred on an almost yearly basis for days to months at a time.
  - Dam was inspected regularly with no signs of distress.
  - Seepage near the foundation was always clear and did not change flow rate with reservoir level.
  - Bulged area on left abutment remained intact after failure.

Discussion

- Field Evidence Supporting Overtopping Erosion Failure Mode
  - It is unlikely rock apron or groin area rock exposures could sustain the level of overtopping flow reported without significant erosion.
  - The left abutment colluvium in the groin area contains large boulders 10’s of feet across. These likely protected the area and kept that part of the dam intact.
  - Native foundation materials on site are obviously highly erodible.
    - Break apart by hand.
    - Over 20 ft of incision into underlying material since the dam’s failure.
    - Erosion in left spillway during low flow event.

Discussion

- Contributing Factors to Overtopping Erosion Failure Mode (resulted in a weak dam subject to failure)
  - Design and construction quality questionable.
  - Use of minimal mortar to join the conglomerate blocks of the downstream masonry wall and possibly the rock apron.
  - Construction during the winter months.
  - Minimal thickness of the upstream and downstream masonry walls.
  - Placement of a loose rubble fill between the walls.
  - Placement of masonry walls on materials of variable strength.
  - Early settlement, cracking, leakage and erosion may have weakened the dam.
  - Possible build-up of water pressures behind downstream masonry wall from water flowing into cracks on dam crest leading to movement of wall.

Discussion

- Most Likely Overtopping Erosion Failure Mode Progression
  - Water rushing over top of dam and center spillway begin to erode the material along the right groin and at the toe of the dam (either a rock apron or unprotected bedrock).
  - Scour hole grows deeper until it reaches the base of the downstream masonry wall.
  - Bedrock below the masonry wall is eroded and support is lost.
  - Poorly mortared blocks begin to break apart and fall into scour hole.
  - Loose rubble fill is quickly removed and support of the upstream masonry wall is lost.
  - Upstream masonry wall topples and breaks apart resulting in the complete breach of the dam.

Conclusions

- The overwhelming amount of evidence indicates that the failure of Castlewood Dam was caused by scour of the rock apron (if present) and erodible bedrock at the toe of the dam undermining the foundation of the downstream masonry wall during an overtopping event.
- The historical records research and literature review generally showed without a reasonable doubt 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 flood, and that there was no significant evidence to support any failure mode other than that summarized above.

Conclusions

- Accounts of the failure of Castlewood Dam indicate that a heavy rainfall in the drainage basin for Cherry Creek resulted in a flash flood into the Castlewood Reservoir. The water level in the reservoir rose very quickly, eventually leading to overtopping of the crest of the dam by 1.34 ft. Failure of the dam occurred quickly once it was overtopped. According to the dam tender on site that witnessed the failure, the time from initial overtopping to breach of the dam was less than 45 minutes.
- Although there is some evidence indicating that erosion, settlement and cracking may have been problematic, particularly in the first 12 years of the dam's life, it appears that these problems were mostly repaired and only possibly contributed to failure by weakening the dam rather than directly causing its failure.

Conclusions

- Comparisons of expected stream power vs. the erosion resistance of the bedrock below the toe elevation indicate that scour depths would have been deep enough to undermine the foundation of the downstream masonry wall. Once the downstream masonry wall failed, the progressive failure of the un-grouted masonry fill would have occurred quickly followed by the collapse of the upstream grouted masonry wall due to lack of support.
- Use of the Erodibility Index Method for scour analysis was a valid predictor of scour during the overtopping event that occurred at Castlewood Dam.