DATA:

26 March 1979, Major repair (MR)

World Register of Dams
U.S. Register of Dams

Height
Reservoir
Purpose
Geology

DESCRIPTION OF DAM:

EAST FORK DAM

East Fork of Tuckasegee River
Tuckasegee, North Carolina
1955, Rockfill
1973, page 407, line 3
1963, page 104, line 17
135 ft (41.1 m)
11,400 acre-ft (14,100 x 10^3 m^3)
Hydroelectric power
Granite

EAST FORK DAM is rock filled with an earth face. The dam (41.1 m) high with a crest length of 385 ft (117 m). The primary purpose is hydroelectric power.

DESCRIPTION OF INCIDENT:

1955, Accident
1965, Accident, Type 2 (A2)
1982, Accident, Type 1 (A1)

Cause - Granite foundation cracked during construction and worsened with subsequent freeze/thaw action

BIBLIOGRAPHY:

None

CONTACT:

Nantahala Power and Light Co.
Franklin, North Carolina
28734

East Fork Dam
"firm foundation" rock. For over 3,000 feet (900 m) of the embankment length, it was decided that the "firm foundation" rock was too fractured, so a secondary cutoff trench was excavated from 5 to 7 feet (1.5 to 2.1 m) into the fractured rock. A single-row grout curtain was developed from a grout cap constructed from the bottom of the cutoff trench.

DESCRIPTION OF INCIDENTS:

Incident 1:

In May 1965, Fontenele Reservoir was filling for the first time and seepage and sloughing was observed in the backfill on the left side of the spillway (which is near the right side of the embankment). A drain pipe was installed along the spillway to remove the seepage, but the sloughing continued. In early September 1966, the reservoir was about 2 feet (0.6 m) over the spillway crest and seepage increased rapidly from about midheight of the dam near the right abutment. The seepage rapidly eroded about 10,500 yd³ (8,000 m³) of embankment; loss of the total reservoir was prevented only through high releases from the outlet works.

The erosion at Fontenele Dam was believed to be caused by the rock conditions at the site and their treatment. Large, up to 6 inches (150 mm), stress relief joints in the left abutment that were partially filled with natural debris were treated by grouting. It is believed that the grouting was not fully effective in this area both because of the natural debris in the joints and because grouting cannot realistically be expected to work perfectly. It is also believed that the pressures from grouting in the right abutment area may have split open some of the existing rock layers. Surface treatment at the embankment-abutment contact, in addition to the grouting, could have been beneficial.

Incident 2:

Following the completion of embankment repairs in 1966, the reservoir was operated normally for a number of years. In late 1982, a small amount of previously unobserved seepage was seen at the toe of the dam near both the left abutment and the central portion of the dam. Study of the design and construction of the embankment indicated inadequate foundation treatment and the reservoir was restricted in 1983 to 10 feet (3 m) below normal. Investigation of the left side of the embankment in 1983 indicated there were numerous very soft areas near the embankment foundation contact. As a result of finding these soft areas, the reservoir was restricted to 25 feet (7.5 m) below normal and consideration of repair alternatives was begun.

In early 1985, it was observed that the water pressure in a piezometer, near the central portion of the embankment where seepage had been noticed in 1982, had risen over 10 feet while the reservoir remained constant. As the pressure in the piezometer continued to rise, investigations indicated it was valid, the decision was made to lower the reservoir an additional 38 feet (11.6 m), making it 63 ft (19.2 m) below normal water surface elevation.

REMEDIAL MEASURES:

Incident 1:

The reservoir was lowered as rapidly as possible. The eroded portion of embankment that needed to be replaced was 345 feet (105 m) long at the crest and 65 feet (19.8 m) high. The foundation treatment prior to replacing the embankment consisted of 1) redoing a portion of the original left and right abutment grout curtains and extending them further into both abutments and 2) blanket grouting the upstream rock beneath the failed embankment area.

Incident 2:

Reservoir restrictions were initiated. Following the conclusion that the foundation treatment for Fontenele Dam was such that a failure from piping could occur anywhere along the length of the dam, construction alternatives were considered. The alternative selected was an unreinforced continuous concrete cutoff wall that would extend essentially for the length of the dam from near the crest down through the embankment and through the fractured foundation rock. The 2-foot (0.6-m) wide cutoff wall is being constructed by excavating the embankment and rock with a rock mill that supports the excavation with slurry. Following excavation of the length desired for a segment of the wall, generally about 30 feet (9 m), the slurry is displaced by concrete starting from the bottom of the trench. A successful test section was constructed in 1985-86 and the repairs should be completed in 1989.

BIBLIOGRAPHY:

"Final Safety Evaluation of Fontenele Dam", Seedskadee Project, Wyoming, Bureau of Reclamation, Denver, Colorado, July 1984


CONTACT: Bureau of Reclamation
Engineering and Research Center
PO Box 25007
Denver, Colorado 80225-0007
from being overtopped, the outlet works and powerhouse were operated at full capacity and both spillways were used from June 2 to July 23, 1983. The operation of the waterways is summarized in figure 1. Within 3 days after the left spillway began operation, rumbling noises were heard from the spillway tunnel and portions of the tunnel lining were seen in the discharging water. The left spillway gates were then briefly closed for an inspection of the tunnel and the right spillway continued operation. The inspection revealed that cavitation and erosion damage were taking place in the vertical bend at the lower portion of the tunnel with approximately 50 yd3 (35 m3) of concrete lining removed. Flood flows continued to fill the reservoir requiring both spillways to be operated continuously for about 2 months with a peak discharge of 32,000 ft3/s (905 m3/s) in the right spillway. After the flood inflows had subsided, the spillway gates were closed and the tunnel de-watered. Damage in the left spillway consisted of a hole 35 ft (10.7 m) deep, 134 ft (41 m) long, and 50 ft (15 m) wide excavated into the sandstone at the downstream end of the vertical bend, extensive damage to the concrete lining upstream from the "high hole," removal of three-fourths of the tunnel liner circumference in the immediate area, and deposition of a large amount of the excavated material at the downstream end of the tunnel. Damage in the right spillway was less extensive than in the left spillway and primarily consisted of a hole, 15 ft (4.6 m) deep, and 12 ft (3.6 m) long, at the downstream end of the vertical bend.

REMEDIAL MEASURES:
Remedial measures taken for the spillway damage at Glen Canyon Dam consisted of two parts:
- Measures taken during spilling to minimize damage to the spillways and protect the safety of the dam.
- Measures taken after spilling to ensure the spillways would not be damaged during future spillway operations.

Once it was known that the spillways were being damaged by cavitation and erosion, spillway operations were developed that minimized the discharges through the spillways, especially the right spillway. The powerhouse and river outlets were operated at near capacity for the duration of the flood. Eight-foot (2.4 m) high steel flashboards were added to the top of the spillway gates to permit smaller releases through the spillway, increase flood scourage storage in the reservoir by 1.75 x 106 acre-ft (1.5 x 109 m3), and allow the spillway gates to be closed several weeks early so spillway repairs could begin as soon as possible. Spillway discharges were kept as low as possible in the right spillway to limit damages because the area of damage in the right spillway was further upstream than that in the left spillway. It was
felt there was a greater chance of damage in the right spillway propagating upstream far enough to allow an uncontrolled release of the reservoir. The right spillway was also being kept in reserve in case the flood forecasts were in error or a large thunderstorm was to occur.

Once the spillway gates were closed on July 23, 1983, construction activities began on repairing the tunnels to ensure they would be available should another flood occur in 1984. To prevent cavitation damage from occurring during future spillway operations, the construction activities of late 1983 and early 1984 included the addition of aeration slots in the upper portion of each spillway tunnel. The successful design and construction of the aeration slots and tunnel repairs were verified in prototype tests conducted in August 1984.

BIBLIOGRAPHY:


Burgi, P. H.; Eckley, M. S.: “Repairs at Glen Canyon Dam,” Concrete International: Design and Construction, American Concrete Institute, Detroit, Michigan, March 1987.

CONTACT: Bureau of Reclamation
Engineering and Research Center
PO Box 25007
Denver, Colorado 80225-0007

Figure 1. - Operation of waterways - May/August 1983
DESCRIPTION OF DAM:

The dam was constructed in 1975. Reservoir releases are made through a 42-inch (1,050-mm) diameter reinforced concrete conduit with a 30-inch (760-mm) reinforced concrete pipe drop inlet spillway and a low-level slide gate. The slide gate, located within the wet well of the inlet structure, is a 24- by 30-inch (610 by 760-mm) seating head type, flat-backed, heavy duty Armco sluice gate with a non-rising, 1.125-inch (28.5-mm) diameter, stem. The gate is operated with a "T"-handled, square head, winch. Mounted in the wall between the wet and dry well, 28 ft (8.5 m) above the gate, is a 3-inch (76-mm) drawdown gate valve which is also operated with the "T"-winch. Running into the base of the dry well is a 6-inch (150-mm) pipe covered by a flap gate. The normal pool elevation is elevation 759.6. The invert of the gate is at elevation 719.5.

DESCRIPTION OF INCIDENT:

The slide gate was operated satisfactorily prior to the initial filling of the dam in 1975. The slide gate was opened when the pool was at normal pool elevation for shoreline maintenance in 1976. When the gate was closed, the stem buckled. The stem was replaced, but again buckled on closure of the slide gate.

REMEDIAL MEASURES:

Sand bags were placed in the wet well to temporarily seal the opening when the slide gate would not close. The slide gate and stem were replaced with self-contained slide gate and stem having a minimum length of stem in compression.

GLEN FLINT LAKE DAM
1975, N/A
1976, Major repair
Cause - Inadequate design
DESCRIPTION OF INCIDENT:

Piezometer readings and observations of relief well flow indicated higher than anticipated uplift pressures at the downstream toe in some areas. Subsequent sub-surface explorations revealed a mass of permafrost creating a cut-off just downstream of the main embankment.

A plan view of the project and typical cross sections in permafrost areas are attached.

REMEDIAL MEASURES:

No emergency measures were taken. The design of additional relief wells between the downstream toe and the 'permafrost cut-off' is underway.

To avoid such incidents in the future, extensive explorations will be required in areas of discontinuous permafrost. The abrupt changes in vertical and horizontal boundaries of permafrost masses within the unfrozen soils must be defined.

BIBLIOGRAPHY:


CONTACT: Corps of Engineers
Alaska District
P.O. Box 398
Anchorage, Alaska 99506-0898

DATA:

1963-1985, Major repair (MR)
World Register of Dams
U.S. Register of Dams
Height
Reservoir
Purpose
Geology

NAVAJO DAM
San Juan River
Farmington, New Mexico
1963, Zoned earthfill

Significant increase in seepage and wetting of downstream shell
1984, page 30, line 14
1963, page 140, line 4
402 ft (122.5 m)
1,708,600 acre-ft (2,108 x 10^6 m^3)
Irrigation
Bedrock at Nava Jo Dam is sedimentary in origin and is nearly flat lying with no folding or faulting. The rock consists of sandstones, with interbeds of siltstones and shales. The sandstones are moderately to highly permeable. Weathering, in the form of iron staining, extends about 200 ft (65 m) deep into both abutments. On the right abutment, weathering is more intense increasing the permeability of the rock. Shale and siltstone beds range from thin laminae to beds 20 feet (6.1 m) thick. The shale and siltstone beds are unaffected by weathering and essentially impermeable, directing water laterally into the more porous overlying sandstone and along intermittently open bedding planes.

As a result of river downdcutting, joints and cracks have formed in the rock parallel to the canyon walls. These open joints are present in both abutments, but are more extensive in the left abutment. Also, open horizontal bedding planes exist intermittently along the contacts between the sandstone and shale beds of both abutments.
The installation of a material at Navajo Dam is predominantly lean to sandy clays and silts of low plasticity. Zone 1, the impervious core of the dam, consists of a mixture of coarse sand, gravel, and cobbles. Zone 3 consists of a mixture of clays, silts, sands, and gravels not suitable for use in zones 1 and 2. The upstream face of the dam has a 3-foot (0.9-m) layer of riprap which provides slope protection from the crest to elevation 5985 feet (1824 m).

DESCRIPTION OF INCIDENT:

Seepage at Navajo Dam was first observed on 5 June 1963, about 1 year after the outlet gates were closed for the initial filling of the reservoir. Seepage has increased during the life of the dam, with total measured seepage from both abutments currently averaging about 1,300 gal/min (6.8 m³/min).

Studies indicate there is a high probability of seepage flowing along the embankment-abutment contact which would erode the core material into the untreated joints and cracks. This erosion will eventually cause failure of the structure. For this erosion or piping to occur, several conditions must exist: (1) the embankment core material must be erodible; (2) the flow rate along the interface must be sufficient to cause erosion; (3) jointing and cracking of the abutment rock. Because removal of the major system of near-vertical joints would have required extensive rock excavation, it was determined during construction that excavating only the overburden on the abutment and treating the joints by blanket grouting through risers would be more appropriate. Embankment material was placed against the abutment surfaces immediately after excavation. The cracks and joints were then grouted by riser pipes extended up through the fill to reduce foundation uplift and movement. The sealing of near-surface cracks with this type of grouting, which was practiced at the time the dam was constructed, is not considered adequate by present-day standards. Also, with this type of grouting operation, it is difficult to assess the effectiveness of the near-surface joint seal obtained.

REMEDIAL MEASURES:

Left Abutment Concrete Diaphragm Cutoff Wall - The installation of a 2.7-ft (0.8-m) thick wall would extend vertically from the crest of the dam into the foundation rock. This wall will cut off all seepage at or near the abutment contacts.

Right Abutment Drainage Tunnels - A modification alternative involving the installation of a tunnel and a drain system into the right abutment at Navajo Dam was selected. The tunnel and filtered drain system essentially eliminates the potential for uncontrolled piping by directing the seepage away from the embankment-abutment contact.

The drainage system will reduce the saturation of the downstream shell of the embankment by intercepting and controlling the seepage. This will increase embankment stability to an acceptable level and remove the threat of dam failure from an unstable embankment.
The right abutment drainage scheme would include installation of a 1,045-ft (318.5-m) long, 10-ft (3,000-mm) diameter tunnel with 11,700 ft (3,566 m) of 4-inch (100-mm) diameter drain holes and a groin drain.

**BIBLIOGRAPHY:**

None

**CONTACT:** Bureau of Reclamation
Engineering and Research Center
PO Box 25007
Denver, Colorado 80225-0007

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**NEWTON FALLS PAPER MILL LAGOON NO. 4**

**Barrel Creek**

Newton Falls, New York

**1965, Earthfill**

**DATA:**

6 October 1984, Failure, Type 2 (F2)

Waste treatment pond for a paper mill.

**DESCRIPTION OF DAM:**

The dike for the Newton Falls Paper Mill Lagoon No. 4 is approximately 30 ft (9.1 m) high with a crest length of 300 ft (91 m). The dike is a zoned earthfill with an impervious core and sand filter. Both faces of the dike slope at 2.5:1 (H:V). The top of the dike is 13 ft (4 m) wide. The lagoon is a waste treatment pond for a paper mill.

**DESCRIPTION OF INCIDENT:**

When it became necessary to lower the lagoon to install a flow monitoring system, it was discovered that no means had been provided to drain it. Therefore the following steps were taken:

1. The original drawings were checked to verify the type of embankment construction.
2. The construction was also checked with the mill yard supervisor when the dam was constructed. He verified that clay was used in the construction of the dam.

The procedure decided upon to lower the level of the dike was to slowly skim off the horizontal top surface of the dike for a length of approximately 60 ft (18.3 m) so that a wide spillway with minimum flow at any one point was created. This was considered to be safe as the dike was constructed with an impervious core. Also, the dike was considered to be heavily compacted as the crest of the dam was actually used as a roadway for heavy trucking for many years.

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**NEWTON FALLS PAPER MILL LAGOON NO. 4**

**1965, Earthfill**

6 October 1984, Failure, Type 2

Cause - While a spillway was being cut in the embankment to lower the reservoir, the outflow eroded the embankment.