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Dam-Breach hydrology of the Johnstown flood of 1889–challenging the findings of the 1891 investigation report

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Abstract

In 1891 a report was published by an ASCE committee to investigate the cause of the Johnstown flood of 1889. They concluded that changes made to the dam by the South Fork Fishing and Hunting Club did not cause the disaster because the embankment would have been overflowed and breached if the changes were not made. We dispute that conclusion based on hydraulic analyses of the dam as originally built, estimates of the time of concentration and time to peak for the South Fork drainage basin, and reported conditions at the dam and in the watershed.

We present a LiDAR-based volume of Lake Conemaugh at the time of dam failure (1.455×10^7 m^3) and hydrographs of flood discharge and lake stage decline. Our analytical approach incorporates the complex shape of this dam breach. More than 65 min would have been needed to drain most of the lake, not the 45 min cited by most sources. Peak flood discharges were likely in the range 7200 to 8970 m^3 s^{-1}. The original dam design, with a crest ~0.9 m higher and the added capacity of an auxiliary spillway and five discharge pipes, had a discharge capacity at overtopping more than *twice* that of the reconstructed dam. A properly rebuilt dam would not have overtopped and would likely have survived the runoff event, thereby saving thousands of lives.

We believe the ASCE report represented state-of-the-art for 1891. However, the report contains discrepancies and lapses in key observations, and relied on excessive reservoir inflow estimates. The confidence they expressed that dam failure was inevitable was inconsistent with information available to the committee. Hydrodynamic erosion was a likely culprit in the 1862 dam failure that seriously damaged the embankment. The Club's substandard repair of this earlier breach sowed the seeds of its eventual destruction.

Keywords: Earth sciences, Hydrology

1. Introduction

On May 31, 1889, between 2:50 and 2:55 p.m., the South Fork dam breached, releasing the torrent now known as the Johnstown Flood (Kaktins et al., 2013). More than 2200 lives were lost and hundreds of the victims were never recovered. An outstanding historic account of the flood was written by McCullough (1968). In the aftermath, on June 5th, 1889 a committee led by the esteemed James B. Francis was appointed by the American Society of Civil Engineers (ASCE) to visit the South Fork dam and investigate the cause of its failure. Francis was a hydraulic engineer best known for his work in flood control, turbine design, canal work, dam construction, and weir discharge calculations. He was a founding member of the ASCE and served as its president from November, 1880 to January, 1882. Other committee members included Max J. Becker, ASCE President in 1889, Alphonse Fteley, ASCE Vice President, and William E. Worthen, a past president of the ASCE. The committee visited the South Fork dam and downstream locations, reviewed the original design of the dam and subsequent modifications made during repairs, commissioned an elevation survey of the dam remnants, interviewed eyewitnesses, and performed various hydrologic calculations. In the end they determined the dam would have failed even if it had been maintained within the original design specifications.

The committee concluded that:

> "The [South Fork] Hunting and Fishing Club [sic], in repairing the breach of 1862, took out the five sluices [drainage pipes] in the dam, lowered the embankment about 2 feet, and subsequently, partially obstructed the wasteway [spillway] by gratings, etc., to prevent the escape of fish. These changes materially diminished the security of the dam, by exposing the embankment to overflow, and consequent destruction, by floods of less magnitude than could have been borne with safety if the original construction of 1851–1853 had been adhered to; but in our opinion they cannot be deemed to be the cause of the late disaster, as we find that the embankment would have been overflowed and the breach formed if the changes had not been made. It occurred a little earlier in the day on account of the changes, but we think the result would have been

equally disastrous, and possibly even more so. . . . " (Francis et al., 1891, p. 456).

This claim that the dam, even as originally constructed, would have failed bears scientific scrutiny. We have analyzed the time of concentration (t_c) for the drainage basin and flood inflows to the lake on May 30–31. We examined whether two spillways (an original auxiliary spillway on the southwest abutment was missed by the committee) and the drainage pipes together, along with greater storage capacity behind a higher impoundment, could have prevented overtopping of the dam if it had not been lowered as much as 0.9 m when the South Fork Fishing and Hunting Club (SFFHC) rebuilt the dam. Our analysis is supported by river level observations that stream inflows to the lake had peaked hours before the dam breach.

Our research relies on many 19th century publications and more recent work that document historic data using English units rather than SI units. For most of our calculations we use the always preferable SI units, but where we highly depend on old data sources we report the original English units. We believe this approach will help confirm our appropriate use of the 19th century data and will aid future workers who may further analyze this dam breach disaster.

2. Background

2.1. Changes made to the South Fork dam

The South Fork dam and its impoundment, Lake Conemaugh (also known as the Western Reservoir), were located near the present-day small towns of St. Michael and Sidman, in Cambria County, Pennsylvania (Fig. 1). The impoundment was originally built by the Commonwealth of Pennsylvania to supply water during low-flow periods by way of the Little Conemaugh River to the downstream canal system in Johnstown. The design and construction history of the South Fork dam and reservoir are discussed by Francis et al. (1891) and reviewed by Kaktins et al. (2013). A view of the dam in cross section is shown in Fig. 2. After the state of Pennsylvania sold the canal properties to the Pennsylvania Railroad in 1857, the dam and reservoir were of no direct use to the railroad and were left with minimal oversight. In July of 1862 the dam breached gradually, draining the lake in half a day. The history of this breach is further discussed in the file of online supplementary material. The Pennsylvania Railroad no longer needed the dam and in 1875 they sold the land parcel including the dam and former lake to John Reilly, a former congressman from Altoona. In 1879 Reilly came to an agreement with Benjamin Ruff for transferring the property, but the formal sale was directly to the SFFHC in March 1880 (McGough, 2005). Ruff intended to establish a resort. But the 1862 failure set the stage for the dam breach disaster in 1889 because when the dam was rebuilt by the SFFHC significant changes were made to the original
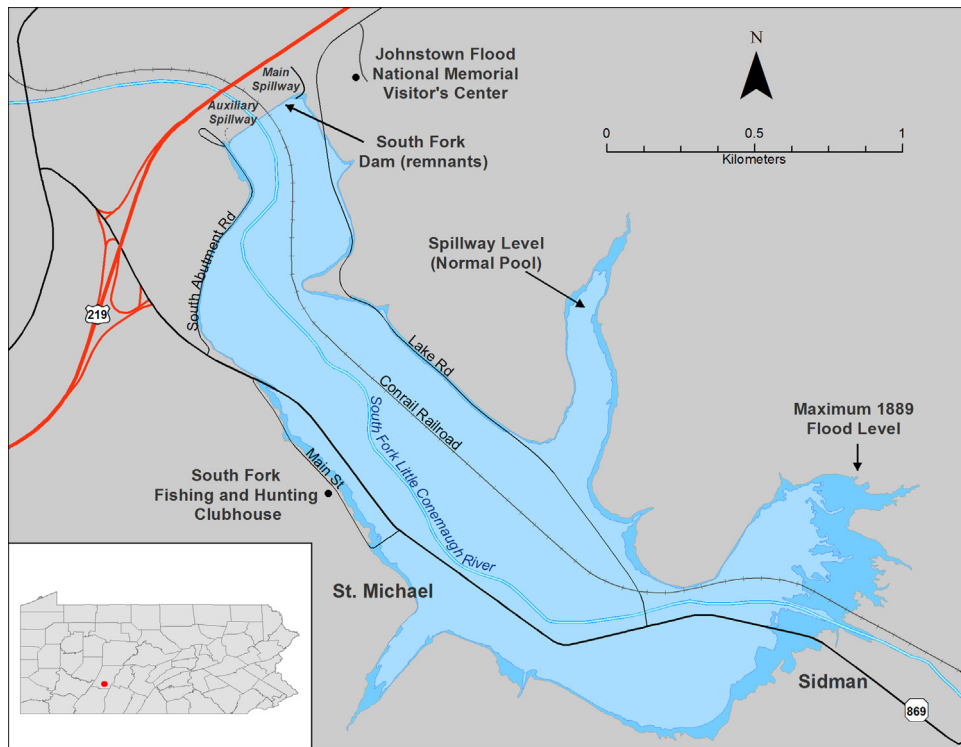
Fig. 1. Location map of the South Fork dam and Lake Conemaugh. The towns of St. Michael and Sidman and the Conrail branch line in the former lakebed did not exist at the time of the flood.

design. The changes, such as lowering the dam crest, the omission of low-permeability puddled clay that had originally been emplaced on the upstream half of the embankment, and the use of mining wastes containing plastic clays, are discussed by Kaktins et al. (2013). A very significant change was the removal of the sluice or discharge pipes at the base of the dam and blocking the opening of the remaining stone culvert portion with hemlock planks. It is unclear when this occurred. McCullough (1968, p. 55) states that although Congressman Reilly sold the property at a slight loss, he made up for it by first removing the old cast iron discharge pipes and selling them for scrap. This statement differs from Francis et al. (1891, p. 445 and 456) who wrote that the SFFHC removed the pipes. They also documented that the dam repairs began in April 1880, several months before the property was conveyed to the SFFHC. In any case, the rebuilt dam now had no mechanism to control the lake water level except for discharge through the main spillway.

Another change relates to the masonry stones that were originally placed on the upstream side ("slope wall") of the dam (Fig. 2). As shown in Fig. 3 of Kaktins et al. (2013), the placement of these dressed stones on the northeastern side of the dam seemed to be incomplete at the time of dam failure. According to Benjamin Ruff, in a written reply to an engineering report from Daniel J. Morrell (Cambria
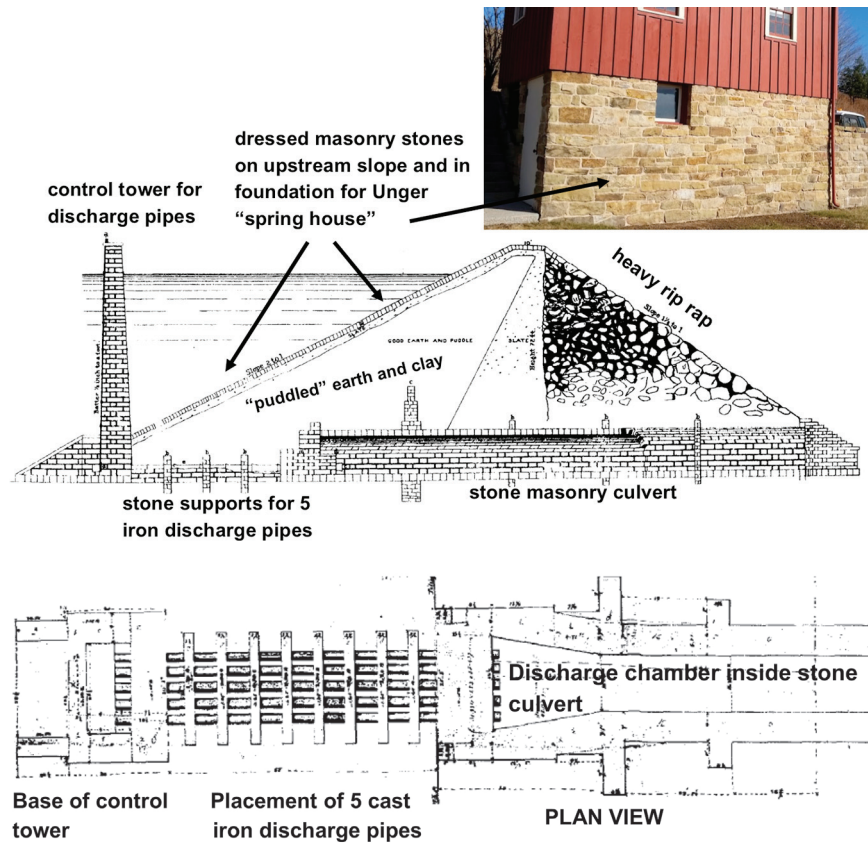
Fig. 2. View of the original design of the South Fork dam (Morris, 1841), showing heavy riprap on the downstream face, a broad "puddled" clay section on the upstream side, slope wall of dressed stone on the upstream face, and a masonry culvert beneath the dam center fed by cast-iron pipes to discharge water and control the lake level. In 1846 the plan was revised to replace the masonry control tower with a wooden frame tower (Francis et al., 1891, p. 444). Modified from Plates LI and XLVIII of Francis et al. (1891). Photo inset at upper right: dressed masonry stones in the foundation of the Unger "spring house" at the Johnstown Flood National Memorial (photo by N. Kaktins).

Iron Company), the face of the dam on the lake side was not covered with riprap, but was covered with a "slope wall" (McGough, 2002, p. 24). Therefore the masonry cover that comprised the slope wall may well have been complete for the original dam, but some of those dressed stones were apparently removed during or before the dam reconstruction. A National Park Service (NPS) photograph reproduced in McGough (2005, Illustration #9) clearly shows that, at least on the northern side, the original dressed stones have been replaced by riprap on the rebuilt dam. We found masonry stones that appear similar in size to the original specifications incorporated in the foundation of the house and spring house of the former F. J. Unger property, on the north side of the dam (Fig. 2, inset top right). Unger was the last president and manager of the SFFHC. The former barn reportedly also had dressed stones in its foundations. They may have been scavenged from the dam before it was rebuilt because photos of the lake side of the

Fig. 3. Large sandstone riprap on the western (left) and eastern (right) remnants of the dam. These represent the downstream side of the dam and illustrate the excellent riprap used in the original construction, obtained mostly from excavation of the main spillway (photos by U. Kaktins). Large riprap like this was *not* used in reconstruction efforts by the SFFHC (see Fig. 4). The person shown stands 1.8 m high.

dam after the flood show that the remaining original slope wall covered less than the bottom fourth of the slope on the northern remnant of the dam. It would certainly have been unusual in the 1800's to fabricate dressed stones to use in the foundations of a barn or spring house, unless such stones were already available. It should be noted that the spring house foundation shown in Fig. 2 is not entirely original. Parts were restored by the NPS to repair damage. The present foundation is likely a good representation of the original, given the NPS's goal of working to accurately preserve historic sites.

One of the key changes was that the large riprap originally used to cover the downstream slope of the dam (see Fig. 3) *was not used* to cover the repaired section. Smaller riprap was used, and this is evident in plate LIIIA of Francis et al. (1891) (also see Fig. 4). Despite this photographic evidence to the contrary in their own report, Francis et al. (1891, p. 446) state that "heavy" riprap was used to cover both sides of the repaired embankment. Also the original dam, instead of a heart wall, relied on "puddled" clay layers to ensure a low permeability integrity of the embankment. But the puddled layers were not replaced during the repairs.

Another major change was the lowering of the dam crest, reportedly to widen the carriage road on top (Francis et al., 1891, p. 446), but the lowering of the crest was also an obvious expedient to quickly get material to begin repair of the partial breach of 1862 (Kaktins et al., 2013). We believe that was the primary reason for
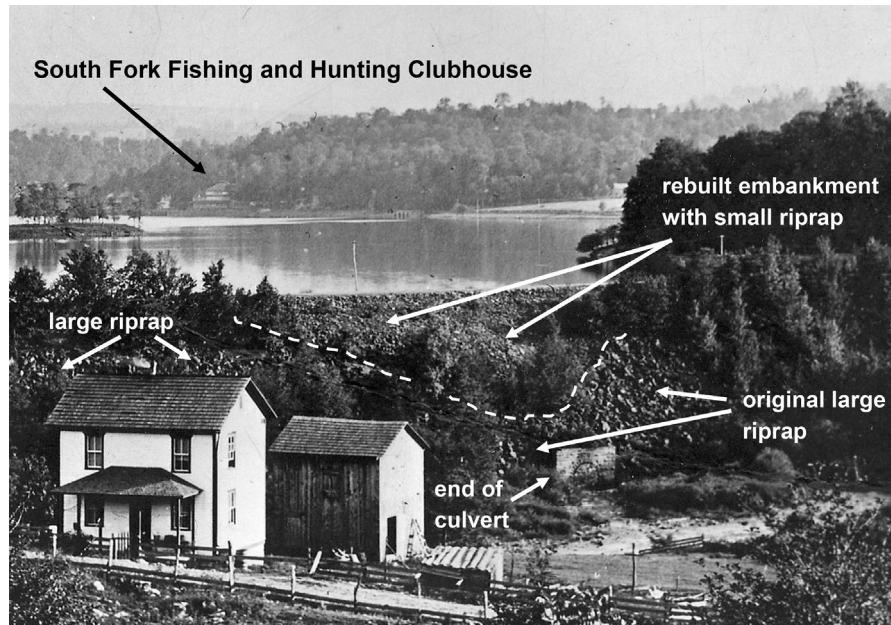
Fig. 4. Undated photo taken after the 1862 partial dam failure but before the 1889 flood. View is toward due south. After the repairs, large riprap is missing from the downstream center of the dam but is clearly seen on the eastern (left) and western (right) sides of the original embankments (see Fig. 3). Note the lesser amount of vegetation on the repaired section (zone above dashed white line). The house site at lower left now lies beneath a bridge foundation on Interstate 219. Base image modified from Plate LIIIA of Francis et al. (1891) and also available from the Johnstown Heritage Association at: http://jaha. org/edu/flood/why/img/dam_gallery/pages/southfork_dam.html. [Accessed 3/2/2016].

lowering the crest. The lowering of the dam crest is nicely shown even today by the fact that large riprap boulders on the intact eastern embankment stand higher than the present-day crest (Fig. 5). Only a few large riprap remain near the crest on the western dam remnant. That area was accessible by road in later years, and we believe most of the boulders along that crest were long ago removed for other purposes. Unfortunately for the SFFHC, the initial material stripped from the dam crest and used to fill the 1862 breach was washed out by heavy rains in December 1879. Additional changes included the construction of a bridge over the mouth of the spillway and the installation of a boom and fish screens at the bridge. These features would have had the effect of somewhat reducing the main spillway discharge capacity.

2.2. Profiles of the dam remnants

An elevation survey of the dam remnants was performed after the flood and was published by Francis et al. (1891). These data were used to construct the profile of dam remnants shown in Fig. 6, which extends southwest from the main spillway along the former dam crest, across the breach, to the western dam remnant and the

Fig. 5. View along crest of the eastern dam remnant, looking westward toward the observation platform. This part of embankment represents original construction. Large riprap from the original construction is *still in place*, standing more than 0.6 m higher than present crest, as a result of the embankment crest being lowered as much as 0.9 m by the SFFHC. White ruler on rock is 0.33 m long (photo by N. Coleman).

hillside beyond. These 19th century data provide the best available measurements to reconstruct the hydrologic conditions at the dam leading up to the dam breach.

Fig. 5 and Fig. 6 provide direct evidence that the dam was lowered more than 0.6 m to as much as 0.9 m by the SFFHC. Francis et al. (1891, p. 446) claim that the dam was lowered 2 ft and report a mean height of 7.96 ft (2.43 m) above the spillway floor for eight points on the crest of the dam remnants. We sought to recreate their mean height and got a value of 7.90 ft using elevation stations 2, 2 + 50, 3, 3 + 50, 9, 9 + 50, 10, and 10 + 50 (see Francis et al., 1891, Plates L and LIII). However, station 10 + 50 should not have been included because it is part of the natural surface of the abutment and not the embankment. It had a height 9.94 ft above the spillway floor, near the height of the original dam crest. Excluding this station yields a mean height above the spillway of 7.6 ft. More importantly, station 9 is 38 m east of the breach margin and stood just 7.07 ft above the spillway floor. It indicates a lowering of the dam of 0.9 m, and this part of the embankment was laterally distant from the dam center where sag due to inadequate compaction of the repaired section may have further lowered the crest. Evidence of sag is discussed in comments and diagrams by P. Brendlinger (in Francis et al., 1891, p. 465) who estimated that the lowest part of the embankment may only have been 6 ft (1.8 m) above the spillway floor. Davis (1889) estimated a sag of at least 0.6 m, while others proposed even greater sag. The lowering of the crest, along with the sag, substantially reduced the originally designed spillway capacity because the relation between spillway and weir depth and discharge is nonlinear. For example,
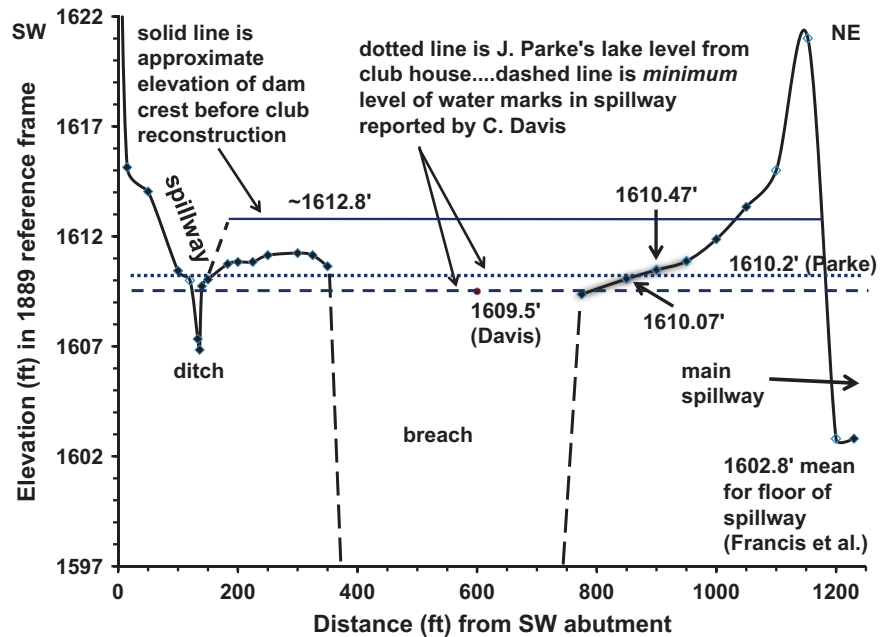
Fig. 6. Profile of the dam remnants and abutments from the post-flood survey in 1889 (Francis et al., 1891). Data are in English units with respect to the 1889 elevation datum. The mean elevation of the main spillway floor is ~1602.8 ft (1889 survey, Francis et al., 1891). The shadowed line from distance 775 ft to 950 ft included a 50 m length of plowed furrow that was preserved after the flood on the NE crest (Unrau, 1980, p. 117; Wellington and Burt, 1889). Overtopping depths would have been greatest at the sagging center. We estimate a lake level of ~1610.1 ft (1889 elevation) at time of dam breach. Data point at the 775 ft distance at the edge of the breach is on a slumped remnant and would not represent the elevation of the rebuilt crest. The bases for water levels by Parke, Davis, and Francis et al. are given in the text. Figure has large vertical exaggeration to show subtle differences in elevations.

increasing the South Fork main spillway flow depth 45%, from 2 m to its original 2.9 m, would raise its discharge capacity 75%.

The profile of the dam remnants has significantly changed since the time of the flood. The two biggest changes were (1) the excavation of the northeastern breach remnant to construct a double line railroad embankment, and (2) the construction of a parking lot at the western abutment. Fig. 7 shows the topographic profiles of the dam remnants, comparing the 1889 topographic data with present-day GPS (global positioning system) profiles. The top frame in Fig. 7 shows the superimposed profiles for the western dam remnant, along with a line that represents the approximate original dam crest. Shown in this way, it is obvious that an auxiliary spillway existed on the western abutment since even after lowering of the crest the abutment is noticeably lower than the embankment. That this area of the abutment was the original location of an auxiliary spillway is further substantiated by observers noting that the initial overflow was in this area (Kaktins et al., 2013), which is why the emergency ditch was dug here. This spillway was shallower than the main spillway, but nonetheless was an important addition to the
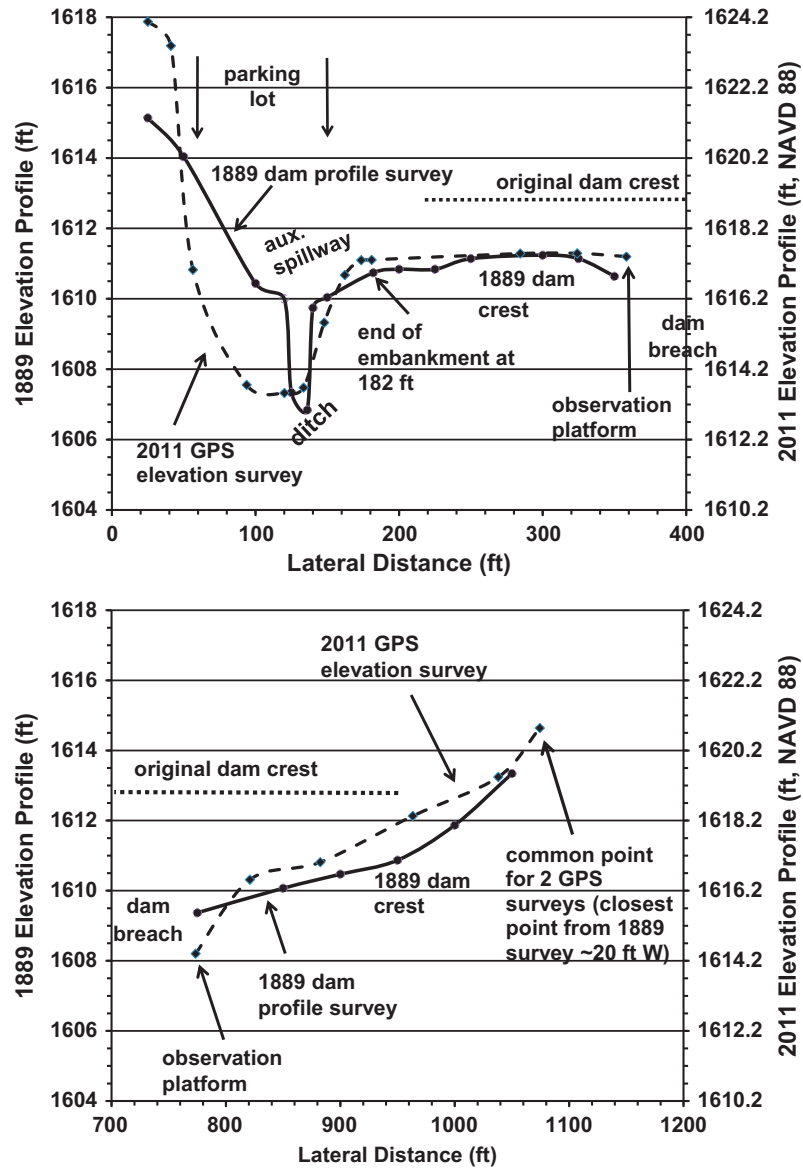
Fig. 7. Topographic profiles of present-day western (top) and eastern (bottom) dam remnants and abutments compared to 1889 survey. There is a difference of ~6.2 ft (1.9 m) between the 1889 and modern elevation (NAVD 88) reference frames; the 1889 data are systematically lower (see Section 2.3.2). Dotted line in each frame shows approximate position of original dam crest.

original dam's outflow capability. A hydrologic analysis of the auxiliary spillway is given in Section 4.2. The ditch dug on the day of the dam breach would not have existed had the dam been rebuilt per the original specifications and therefore is not included in the flow calculations for the original dam.

2.3. Establishing a GPS "benchmark" at the South Fork dam, and finding the difference between the 1889 and modern GPS survey elevations

2.3.1. GPS benchmark

In a contribution to this research, Musser Engineering of Central City, PA established an RTK (Real Time Kinetic) GPS base station near the South Fork dam. This GPS "benchmark" greatly facilitates our research at the dam site and is documented here to support future investigators. This base station is east of the dam beside the footpath that leads from the eastern parking area to the bridge over the spillway. The base station is on a concrete footing with a brass plate (see images in online supplementary material), and the data for this station are given in Table 1. Musser Engineering used an RTK GPS unit at the base station, a TOPCON HiPer Ga, manufactured by Hayes Instrument Co. A laser theodolite was used at the established base to determine coordinates for other features at the site. The Total Station Instrument was a TOPCON GPT 3005W. The absolute elevations obtained by Musser Engineering were further used to calibrate relative GPS measurements collected at the site by Professor Brian Houston of the UPJ (University of Pittsburgh at Johnstown) Engineering Department. To convert these relative data to NAVD 88 (North American Vertical Datum of 1988) we added 13.1 ft. One of the common points in these two GPS surveys was a foundation stone at the base of the dam (see image in online supplementary material).

2.3.2. Difference between the 1889 survey elevations and modern GPS data

Using LiDAR data and results acquired in two GPS surveys at the dam site, we have evaluated the difference between the 1889 and modern-day elevation reference frames. It was challenging to find sites where the surfaces have been relatively stable since 1889. We chose three places at the dam where we are confident the surfaces are little changed, including: (1) the spillway surface beneath the soil layer, (2) station 1050 from the 1889 topographic survey, and (3) the highest points on the western dam remnant.

The spillway surface would seem to be the best location, since it was a bare rock surface in 1889. However, a post-flood photo of the spillway shows that it had an uneven surface with scattered rocks (Francis et al., 1891, Plate XLVI), and over>120 years a soil of leaf litter and colluvium has accumulated. The ASCE topographic survey took measurements directly on the spillway surface. "By our levels [measurements], the floor of the wasteway for 176 feet from the lake averages 1602.82 feet above tidewater" (Francis et al., 1891, p. 446). The locations and number of measurements averaged by the committee were not shown in their report, but they noted the range of elevations varied from 0.14 ft higher to 0.08 ft

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Table 1. Altitude Data for the South Fork Dam (Sources: Musser Engineering, Central City, PA; UPJ Engineering Dept.; and PASDA, 2013a for LiDAR data ("LiDAR" is a portmanteau word combining "light" and "radar").

Location	Altitude NAVD 88 (msl)	Latitude (or Northing)	Longitude (or Easting)	Notes/source
RTK base at JOFL #84	485.606 m (±0.025 m) 1593.1920 ft	N40° 20′ 52.74434"	W78° 46′ 21.93709"	Converted to NAVD 88 using GEOID09 by Musser Eng.
Stone at base of former control tower (see image in additional information)	1544.93 ft			Musser Eng. laser theodolite
Natural surface on eastern dam abutment	1620.90 ft			Musser Eng. laser theodolite
Spillway top of soil surface near 3rd bridge support post from east end	1608.8 ft (1595.66 ft + 13.1 ft)	Northing 371685.331	Easting 1683262.211	UPJ engineering GPS
Natural surface on west side of spillway footbridge	1620.8 ft (1607.738 ft +13.1′)	Northing 371766.455	Easting 1683177.966	UPJ engineering GPS
Soil surface in area of main spillway crest	1611 to 1612 ft			Pennsylvania LiDAR 3.2 ft DEM (see Fig. 8 for map area and scale)

less than 1602.82 ft. They did provide a diagram (top of their Plate LI) showing the profile along the axis of the spillway, with a base level of 1602 ft. It is unclear why they show this number instead of rounding up their measurements to 1603 ft. Another estimate for the spillway floor was given by P. Brendlinger, who took independent measurements a year after the dam breach and gave a spillway elevation of 1603.4 ft (Francis et al., 1891). This was not the spillway surface but was the "top of the [bridge] sill" at a location 26 ft (7.9 m) from the dam end of the bridge.

LiDAR data (PASDA, 2013b) for the surface of the main spillway show that it transitions from ~1609 ft at the upstream entry to a crest of ~1612 ft (491.3 m, NAVD 88) over a distance less than 20 m (Fig. 8). To augment the LiDAR digital elevation model (DEM), which has a vertical accuracy of <1 m, a high-resolution GPS measurement is available for the soil *surface*, at a point 1 m south of the 3rd bridge support post from the eastern end. The GPS elevation is 1608.8 ft (490.36 m, NAVD 88). As part of a 2011 soil survey permit (NPS, 2012) we used a thin penetrator rod to measure soil thicknesses at several points along the axis of the spillway. No soil samples were taken, only depths were determined. The depth of refusal for the thin rod represents a minimum soil thickness. These measurements are used to estimate the elevation of the subsoil spillway surface. At the GPS location the soil thickness is ≥27 cm. Subtracting this, the elevation of the spillway rock surface is ≤490.1 m (NAVD 88). However, this location is near the spillway
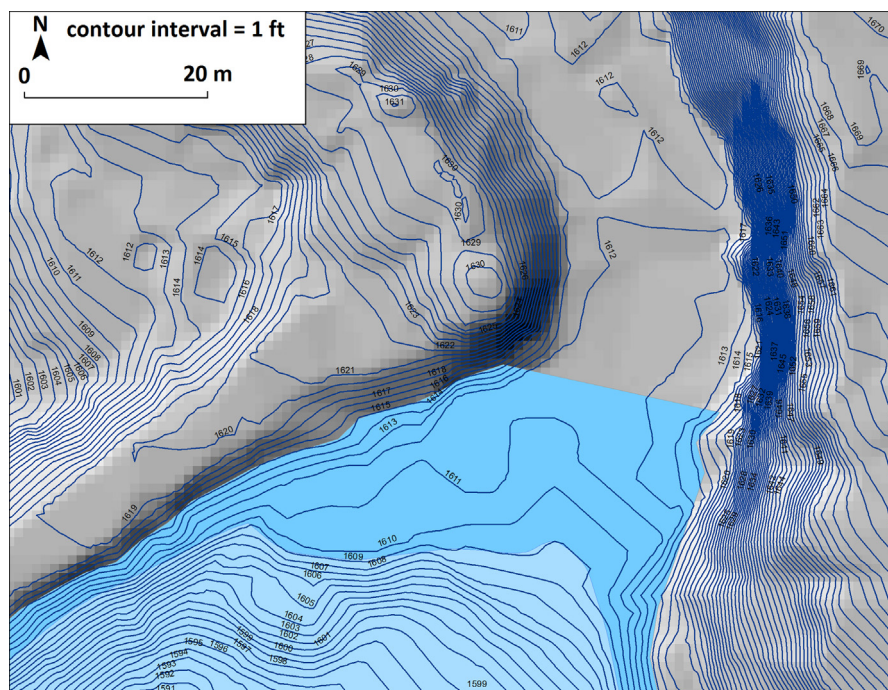

Fig. 8. Contour map of main spillway, produced using LiDAR 3.2 ft DEM of Pennsylvania (PASDA, 2013b). The elevation data for the DEM were obtained before December, 2006.

"lip" and not near the crest of the spillway. Starting from the GPS location, at a distance 35 m to the north along azimuth 12°, three measurements 0.1 m apart with the penetrator rod reveal soil depths of 53 to 56 cm. The narrow range suggests the soil was fully penetrated. The surface elevation here is ∼490.9 m, determined with an inclinometer in two stages over the 35 m distance. Subtracting the soil depths, the spillway surface at the crest has an elevation of ∼490.4 m (NAVD 88). The result is a difference of 6.1 ft between the 1889 survey and modern GPS-based data. Compared to the more accurate GPS measurement, the LiDAR DEM over-predicts the soil surface elevation at the spillway crest by about 0.3 m, which is within the DEM accuracy range. Part of this small over-prediction might be influenced by conditions in the spillway itself, including its vegetated surface, tree-lined upper margins, and steep side slopes. Reports of vertical accuracy testing for LiDAR data published for Pennsylvania are provided by DCNR (2016).

The second location for elevation comparison is on the eastern dam remnant, near the site of an 1889 measurement point, station 1050 (see Francis et al., 1891, Plate L), for which an elevation of 1613.34 ft was reported. A modern GPS measurement near this point yields 1619.44 ft (NAVD 88), equivalent to 493.60 m. The difference between the 1889 and modern reference frames is ∼1.86 m (6.10 ft). This location appears little changed since 1889 because this is the intersection of the embankment with natural ground on the eastern abutment. About 6 m to the northeast a vestige of the old road from South Fork intersects the eastern abutment. High points on the western dam remnant comprise the third site for elevation comparison. As shown in Fig. 7 (top frame), the surface of the footpath on the crest of the western remnant has been leveled since 1889. The three highest points from the 1889 survey are ASCE stations 250, 300, and 325, with elevations in the range 1611.14 to 1611.24, varying only 0.1 ft (3 cm). Two GPS measurements on this same section of crest yield 1617.49 ft (NAVD 88). The difference between the 1889 and GPS reference frames is ∼6.3 ft.

Based on the close agreement of measurements from all three locations, we are confident that the difference between the 1889 and present-day reference frames is in the range from 6.1 to 6.3 ft, with 6.2 ft (∼1.9 m) being our best estimate. This difference was used to construct the dam remnant profiles in Fig. 7.

2.4. Water levels for lake and spillway at time of dam failure

On the day of the dam failure workers ran a plow along the embankment crest to try to raise the lowest part of the crest to delay overtopping. This raised furrow was as much as 0.6 m high in the center of the dam and tapered to about 0.1 m at the ends. Portions of the furrow remained after the dam breach on the eastern remnant and provide evidence that the lake level at the time of dam failure did not exceed the height of the furrow base. Wellington and Burt (1889) describe a preserved

mound near the break and extending for about 46 m on the eastern dam remnant (Fig. 6). The preservation of this soft mound of material, which would have rapidly been swept away by overtopping flows, provides evidence that the lake level was no higher than 1610.1 ft (1889 survey), and may have been somewhat less. Correcting for the difference between 1889 and modern elevations, the maximum lake level was ~1616.3 ft (NAVD 88), or ~492.6 m. Our best estimate of the lake surface elevation at the time of dam failure is in the range from 492.5 to 492.6 m.

Davis (1889) provides important information on water levels in the spillway around the time of the dam breach. After the breach he could not find flood levels on the bridge supports but "Out in the weir [spillway] itself I found marks on the shale banks and on the bushes, showing that the water was over 6 feet above the sill." (Unrau, 1980, p. 128). Davis' degree of uncertainty about the exact height of the water marks above the sill is related to his use of a hand level for the measurements. And it is not clear where on the sill his reference point was located. An additional 8 in (20 cm) for the sill height above floor gives a minimum flow depth of 6.7 ft (2.0 m) in the spillway. Adding this 6.7 ft to Francis et al. (1891) mean floor elevation of 1602.8 ft gives a water level elevation in the spillway of ~1609.5 ft (1889 elevation, Fig. 6). In the present reference frame that elevation is ~1615.7 ft (1609.5 [Davis] + 6.2 ft), or ~492.5 m (NAVD 88). Davis' observations represent the best available information about peak flow depths within the spillway itself, which provides an estimate of critical flow depth over the spillway crest.

Another estimate of spillway flow depth was provided by John Parke, the resident engineer at the SFFHC (Francis et al., 1891), who reported that as the breast was being broadly overtopped the water level was " . . . 7.4 feet above the normal lake level." But that measurement was taken at a stake near the SFFHC clubhouse and represents lake level, not the level in the spillway. Parke did not give the elevation of "normal lake level" or state how it was measured, but this terminology usually refers to the spillway floor as a control on the lake water surface. We interpret it as representing either "top of sill" or the spillway floor elevation, which differ by 0.2 m. If Parke's level was referenced to the spillway floor then there was a 0.2 m difference between his and Davis' measurements. If Parke's measurement related to height "above the sill," then the difference between his and Davis' levels would increase to 0.4 m. These differences are too large to be explained by the slight gradient in the lake surface between the clubhouse and the dam, which would yield a height difference of less than 1 cm. A difference is nonetheless expected, especially when flow in the spillway was deep, caused by the physics of flow over a broad-crested weir. The lake level, i.e., upstream head above the weir crest, will be higher than the critical depth over the crest. Even though observers like J. Parke reported there was no large amount of debris clogging the spillway mouth, additional differences between the lake level and flow level in the spillway would

have been caused by flow resistance through the fish screens and around the bridge supports. The combination of rigid and floating fish screens extended at least 1.5 m up from the bottom of channel. By Brendlinger's estimate the bridge supports and screens may have reduced the cross-sectional area available for flow by 40–50% (Francis et al., 1891, p. 464). Old diagrams of the bridge suggest that the reduction in flow area was not so large.

Wind effects during the storm could have caused intermittent wash-over of the embankment near the time of overtopping. We have not found records on wind speed and direction for the dam vicinity and therefore could not incorporate wind effects in the analysis. The poorly reconstructed dam survived overtopping for>3 h, which may have included some wind-assisted wash-over. In any event, as our analyses will show, a properly rebuilt higher dam would not have been overtopped by lake rise, and hypothetical intermittent wash-overs caused by wind should have had little effect given the robust design and larger riprap.

3. Analysis

3.1. Storage capacity of Conemaugh lake at the time of dam failure

We have developed a new estimate of the storage capacity of Conemaugh Lake using the 3.2 ft DEM derived from Pennsylvania LiDAR data (PASDA, 2013a, b). This DEM has spatial and vertical resolution better than 1 m. LiDAR data are scale independent and permit much more detailed mapping than the previous USGS (U. S. Geological Survey) DEMs that were based on photogrammetric analysis of decades-old imagery. At the moment the dam breached in 1889 the impoundment held about 1.455×10^7 m^3 of water below a contour elevation of 492.56 m, which approximates the modern elevation of the impounded lake at failure. The equivalent tonnage (14.3 million) is less than the usually cited figure of 20 million tons (e.g., McCullough, 1968, p. 41). Previously, Penrod et al. (2006) used a DEM and GIS technology to estimate a lake volume of just over 1.27×10^7 m^3 when the dam failed. Most of the difference between their estimate and ours results from their use of a lower lake stage of 491 m for the time of dam failure (Fig. 1 of Penrod et al., 2006). A stage of 491 m is lower than the present-day soil-covered surface of the main spillway, based on LiDAR and our GPS data. Our new estimate increases the lake volume at the time of dam failure by 15%, a significant amount that would increase the time needed to drain the lake. Our analysis using the LiDAR DEM also shows that the dam *as originally built* with a higher crest would have impounded 1.627×10^7 m^3 (16 million tons) below a lake stage of 493.5 m.

3.2. Storage-elevation curve for lake Conemaugh

Using the LiDAR data we have developed a new storage elevation curve that shows the amount of impounded water beneath any given lake stage (Fig. 9). The shape of the curve shows that small increases in stage height reflect nonlinear increases in lake volume. We have not included a correction for lake-bottom sediments that would have been present in 1889 and would have diminished slightly the original lake volume. There is post-flood photographic evidence that such sediments were present (Fig. 3 of Kaktins et al., 2013), but the average sediment thickness over the basin before the dam breach is not known. A significant fraction of the former lake bottom sediment probably washed away in the first years after the dam breach, before it could be stabilized by vegetation. As part of a former NPS research permit (NPS, 2012), in October 2011 soil samples were collected and analyzed from five locations in the southern part of the lake basin. The samples were collected under the supervision of NPS employees. In this small sample set the soils had an average depth of 27 cm based on depth-of-refusal. Part of this soil depth would have included native soils that were present before the dam and reservoir existed.

3.3. Estimated time of peak inflows to the reservoir

3.3.1. Rainfall data

A key factor for the survivability of the South Fork dam was the time-varying rate of flow into the reservoir. Rainfalls in parts of the region appear to have reached or
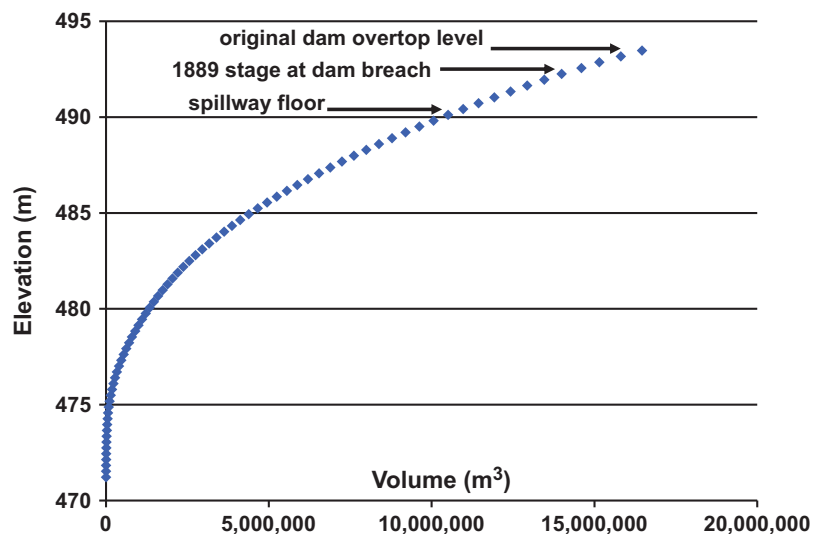

Fig. 9. LiDAR-based storage-elevation curve for Lake Conemaugh. The highest point on the curve represents an overtopping level of 493.5 m for the South Fork dam *as originally built*. Data Source: PASDA (2013a, b).

exceeded the threshold for 100-year, 24-h storms (Fig. B-8 of USDA, 1986). Fig. 10 shows the approximate rainfall distribution in Pennsylvania during May 30–31, 1889. Station data used to develop the map are given in a Table in the online supplementary material. The closest stations to the South Fork dam included Blue Knob (19 km east) and Hollidaysburg (34 km east). The rainfalls associated with the 1889 event were 9 inches (1.00 in = 2.54 cm) or less, with the highest measurements being reported at Grampian (8.60 in), Blue Knob (7.90 in), Charlesville (7.60 in), and far to the east at McConnellsburg (8.99 in). The Blue Knob station lies just east of the watershed. We consider that a plausible average rainfall total over the South Fork watershed was 6 to 7 in, with as much as 7+ in in the easternmost parts of the watershed. There was no precipitation station in the watershed in 1889, but one anecdotal observation was documented by Francis et al. (1891, p. 468). A small dam existed on one of the tributary streams that fed Conemaugh Lake. The proprietor of this dam stated that " . . . a pail, which was empty the night before, had 6½ inches of water collected in it from the rain during
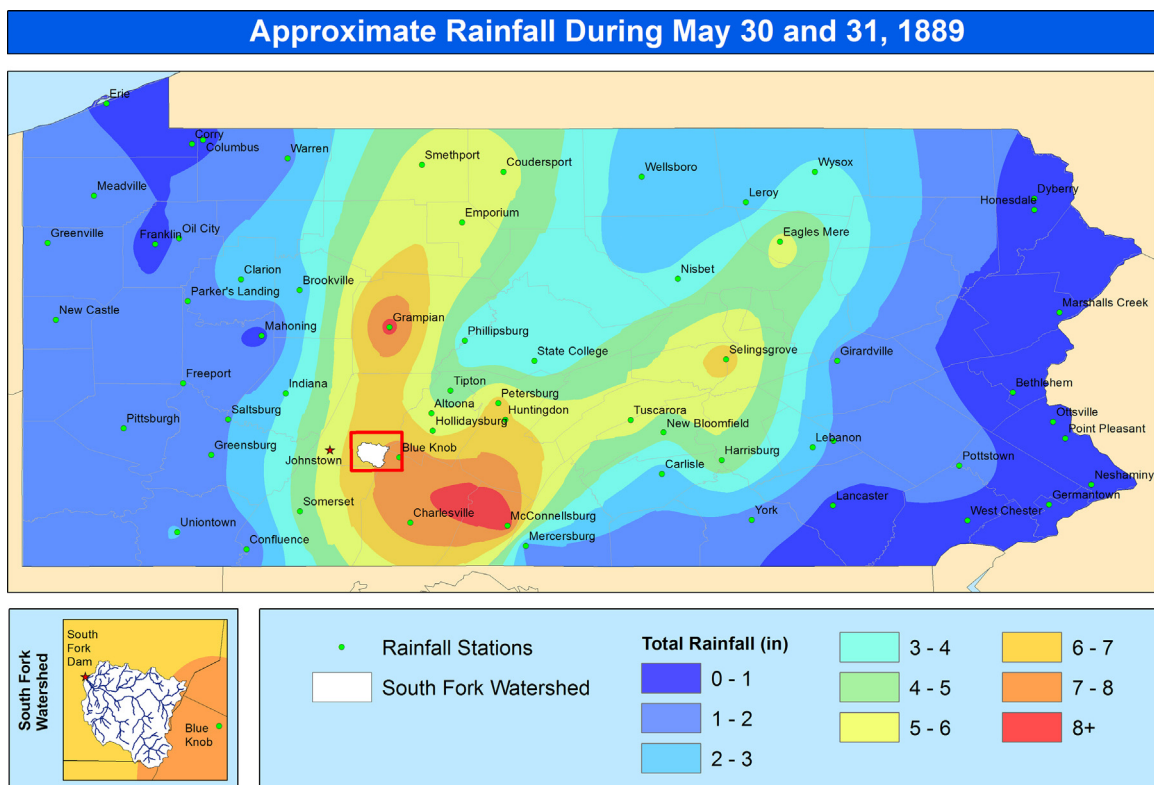

Fig. 10. Approximate rainfall distribution in Pennsylvania during May 30–31, 1889 (data are from Table in online supplementary material). Contour interval = 1 inch of rainfall. Cities and towns on the map are data locations. Contours were developed using the kriging interpolation method with the measured data points. This method of interpolation is based on the statistical relationship between measured points to produce a predictive surface and provides a measure of accuracy in those predictions (ArcGIS 2011) (sources: Blodget, 1890; Russell, 1889; Townsend, 1890).

the night. This was at a point about two miles above the dam." It must be kept in mind that for the actual dam failure only the amount of rainfall up to ∼3 p.m. on May 31 is of primary concern because the dam failed around 2:50–2:55 pm (Kaktins et al., 2013). Russell (1889) estimated that about 75% of the total precipitation had fallen by 3 pm. Although Francis et al. (1891, p. 452–453) were of the opinion that the flow into the reservoir kept increasing up until the time the dam failed, " . . . and no doubt continued to do so for some time longer," the evidence they offer for that claim is questionable. Their analysis is further discussed in Section 4.5.

3.3.2. Time of concentration and time to peak for the South Fork of the little Conemaugh river

We have applied several methods developed in various parts of the U.S. to estimate plausible ranges of time of concentration (t_c) and time to peak (t_p) for the South Fork of the Little Conemaugh River. Time of concentration is the time needed for the whole watershed to contribute to runoff at a downstream point of interest. In practice t_c represents the time for runoff to travel along streams from the most distant part of a drainage basin to a main channel downstream. Where applicable, the Kerby-Kirpich method (Roussel et al., 2005) can be used to estimate t_c. The velocity method (NRCS, 2010) is also commonly used. Both methods estimate t_c as the sum of travel times for discrete flow regimes. The Kerby-Kirpich approach requires few input parameters, is straightforward to apply, and produces readily interpretable results. Resulting t_c estimates are consistent with watershed time values independently developed from real-world storms and their runoff hydrographs (Roussel et al., 2005). Fang et al. (2008) also report that the Kirpich and Haktanir–Sezen methods provide reliable estimates of mean values of t_c variations. Overland flow in the uplands, especially the higher altitude areas of the eastern South Fork watershed, would significantly add to the magnitude of t_c. Therefore, rather than using the Kirpich equation, we prefer the Kerby-Kirpich method which yields a total t_c by adding the overland flow time (Kerby) to the channel flow time (Kirpich):

$$t_c = t_{ov} + t_{ch} \tag{1}$$

where:

t_c = time of concentration

t_{ov} = overland flow time

t_{ch} = channel flow time

Roussel et al. (2005) analyzed 92 watersheds in Texas, and examined various methods for evaluating the time-response of watersheds to rainfall. They

emphasized that none of the watersheds in their database had low topographic slopes. They recommended the Kerby-Kirpich approach as a preferred way to estimate t_c. Their study included watershed areas ranging from 0.65 km^2 to 390 km^2, main channel lengths between 1.6 km and 80 km, and main channel slopes between 0.002 and 0.02 (Roussel et al., 2005). The main channel slope is the change in elevation from the watershed divide to the outlet divided by the length of the main channel and tributaries leading from the divide. For smaller watersheds and intense storms where overland flow is an important component, the Kerby equation can be used:

$$t_{ov} = K(L \times N)^{0.467} S^{-0.235} \qquad (2)$$

Where:

- t_{ov} = overland flow time of concentration (min)
- K = a units conversion factor; 0.828 for traditional units and 1.44 for SI units
- L = the overland-flow length (feet or meters)
- N = a dimensionless retardance coefficient
- S = the dimensionless slope of terrain conveying the overland flow

For the most distant parts of the South Fork watershed near flow divides, the typical length of overland flow is estimated at ~600 m. A dimensionless retardance coefficient (N) of 0.8 was chosen to represent deciduous forest with thick leaf litter. The slope of the overland flow component for uplands near the drainage divides varies considerably, and was estimated to range from 0.1 (i.e., 60 m/600 m) to 0.01. Given these values, the Kerby equation yields a t_{ov} range of 44 to 76 min. These values significantly exceed the 30 min period that is sometimes treated as a "standard" lag time for overland flow.

The Kirpich equation is used to compute the channel-flow component of runoff:

$$t_{ch} = K(L)^{0.77} S^{-0.38} \qquad (3)$$

Where:

- t_{ch} = the channel-flow component of time of concentration (min)
- K = a units conversion factor; 0.0078 for English units and 0.0195 for SI units
- L = the channel flow length (feet or meters), or total basin length minus overland flow length
- S = the dimensionless main-channel slope

The longest channel system in the drainage basin is the main channel of the South Fork of the Little Conemaugh and its tributaries that extend to the southeastern corner of the basin. This main channel path length using a series of straight-line segments is ~17 km long. The extra length of small-scale meanders was not

included because at high flood levels the flow is over the banks and follows more direct paths. A mean channel slope of 0.016 is obtained by dividing the elevation difference between the reservoir basin and the higher channel reaches to the southeast (i.e., 270 m) by the channel path length. These values yield a t_{ch} of 170 minutes; adding the t_{ov} range of 44 to 76 min gives a t_c range of 214 to 246 min, or 3.6 to 4.1 h.

For comparison, the Haktanir and Sezen (1990) method was applied (see Roussel et al., 2005) and yielded a larger $t_c = 4.9$ h. The Folmar and Miller (2008) equation (see NRCS, 2010) is based on the longest hydraulic length in the basin, and was developed using non-urban watersheds up to 52 mi^2. Applied to the larger South Fork watershed the equation yields a lag time of 6.7 h, with lag time being measured from the centroid of excess precipitation to the peak of the hydrograph. For the 52 watersheds studied their equation had an R^2 value of 89%.

We applied an additional method to estimate t_c (in hours) for a basin, by taking the square root of the basin area in *square miles* (Roussel et al., 2005; Fang et al., 2008). This method has no apparent physical basis, but when graphed with a large number of t_c values derived from a group of observed rainfall-runoff analyses, the resulting reference line passes through the general center of the data plot (Fang et al., 2008). The South Fork basin has a total area of 160 km^2, but the drainage area above the former lake basin is less, about 137 km^2 (53 mi^2). Taking the square root yields a t_c of 7.3 h. We consider the value obtained with this approximation, along with the Kerby-Kirpich and other results, provides a practical t_c range of 3.6 to 7.3 h. Roussel et al. (2005) further relate the time to peak discharge (t_p) to t_c. When the Kerby-Kirpich approach is used they advise using the relation estimating $t_p \approx 0.7\ t_c$ for undeveloped watersheds. Applying this empirical relation to the South Fork watershed yields an estimated t_p range of 2.5 to 5.1 h. This relatively small magnitude of t_p is credible for the compact watershed of the South Fork of the Little Conemaugh. Less time is needed for a compact watershed to contribute runoff to achieve a peak discharge rate.

Stream gaging data are not available for the South Fork of the Little Conemaugh River, but due to its frequency and severity of flooding, long period data have been collected for the Little Conemaugh itself (NWIS, 2015). The river has a drainage area of 482 km^2 upstream of the USGS gage at East Conemaugh, Pennsylvania. In response to most regional storms, time of concentration, time to peak, and lag time must be significantly less for the South Fork subbasin, which is far upstream and has a much smaller drainage area. Very large runoff events have been documented for the Little Conemaugh River, the largest being the historic flood of July 20, 1977 with a peak discharge of ~40,000 cfs at East Conemaugh (Brua, 1978). In a 1040 km^2 area north and east of Johnstown, rainfall totals of 6 to 12 inches were measured in 6 to 8 h. Mapping of rainfall showed that the greatest amounts fell

over the South Fork subbasin and also over the Laurel Run subbasin north of Johnstown. Failure of the Laurel Run dam #2 killed 40 people (Brua, 1978, Fig. 3). Six other dams in the region also failed. Brua (1978) estimated a peak discharge in the South Fork of the Little Conemaugh at 24,000 cfs, or 680 m^3 s^{-1} (at Fishertown, just below the former South Fork dam). He also estimated a unit discharge of 456 cfs/mi^2 (5.0 m^3 s^{-1} per km^2) for this subbasin. Brua's (1978) compilation and analysis shows great variability of unit discharge over the subdrainages of the Little Conemaugh River and demonstrates the severe challenge of identifying "design" storms for engineering purposes in basins of this size. The South Fork subbasin makes up only 28% of the Little Conemaugh basin by area, but in the 1977 storm it produced an estimated Q_p>60% of the Little Conemaugh River Q_p as measured at East Conemaugh.

Some insights about time to peak (t_p) for the South Fork channel can be gleaned from the Little Conemaugh gaging data. Land use, terrain slopes near the divides, and the approximate proportions of forested and cultivated lands are similar for the South Fork subbasin and the basin of the main stem, or North Fork of the Little Conemaugh. The streams respond quickly to precipitation events and the time to peak from the time of initial stream response is relatively short. This time interval is a useful approximation of the lag time, which is the interval from the excess precipitation centroid to the peak river discharge. In the extraordinary 1977 flood event, the time to Q_p following initial river level response was <10 h for the Little Conemaugh at East Conemaugh (Fig. 11). The East Conemaugh gage is ~20 km downstream from the former South Fork dam. Therefore the initial response of the stream to precipitation should have been quicker in the smaller subbasin above the dam and t_p must have been significantly shorter, <8h, and probably even less given the estimated range of t_c. This time to peak should represent a reasonable upper limit for the response of the South Fork subbasin to the lesser precipitation amounts in the 1889 storm. Therefore, if the rainfall peak on the night of May 30–31 occurred between 3 and 6 a.m., then local stream discharges would have been expected to peak and start diminishing between 11 a.m. and 2 p.m. (or earlier) on May 31st. That expectation is consistent with local stream observations around noon on that date.

Additional hydrologic research about the Little Conemaugh system was published by Roland and Stuckey (2008). They developed regression equations to estimate flood discharges at various recurrence intervals for ungaged streams in Pennsylvania with drainage areas less than 2000 square miles. They developed these equations using gaging data from 322 monitoring stations in Pennsylvania and adjacent states. They noted that the regression equations are not valid for larger drainage basins or basins with substantial mining activity or regulation by dams. The basin of the Little Conemaugh River has several dams with limited regulation, and extensive underground mining activity that could affect base flow conditions
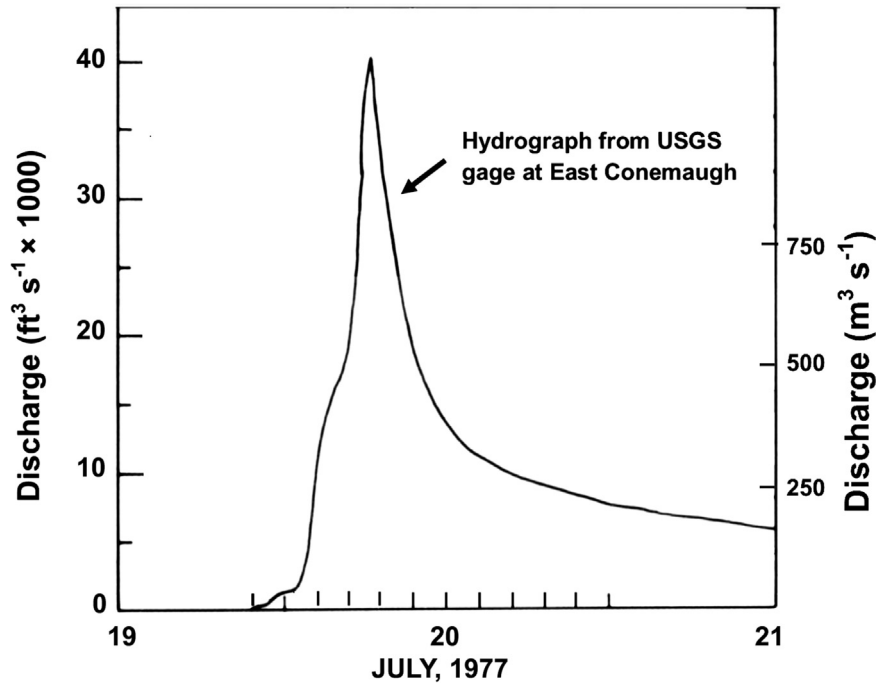
Fig. 11. Discharge hydrograph for the Little Conemaugh River at East Conemaugh during the Johnstown flood of 1977 (after Brua, 1978). In this extreme runoff event, the discharge rate at this gage downstream on the main stem of the river exceeded 50% of its peak for only 5 h.

but would not likely affect storm runoff. But this basin has a long period of gaging dating back to 1936, and these data were used to describe the recurrence intervals for the flood discharges in Table 2. It is unclear how the 500 yr *observed* estimate was derived for Table since the highest discharge rate recorded at East Conemaugh in the last 80 yr was 40,000 cfs during the 1977 flood.

In summary, based on the above discussion we obtain a practical range of t_c for the South Fork subbasin of 3.6 to 7.3 h, and t_p (similar to lag time) in the range of 2.5

Table 2. Little Conemaugh River flood-flow magnitudes for various recurrence intervals computed from observed streamflow-gaging data at East Conemaugh, with predictions from regional regression equations, and a weighted average for gaging stations used in the analysis (from Roland and Stuckey, 2008, p. 50).

USGS gaging station 03041000	Type	Flood-flow estimates (cfs) [1 m^3 s^{-1} = 35.3 cfs]					
		2 yr	5 yr	10 yr	50 yr	100 yr	500 yr
	Observed	10,400	16,800	22,100	36,800	44,500	66,500
	Predicted	11,700	17,900	22,600	35,300	41,700	59,500
	Weighted	10,400	16,900	22,100	36,600	44,100	65,700

to 5.1 h. Data for the 1977 flood from the stream gage at East Conemaugh suggest that t_p in extreme events upstream on the South Fork would be <8 h, and since t_p will normally be less than t_c, we expect t_p would be <7.3 h.

3.3.3. Stream level observations

Kaktins et al. (2013) document observations about local rainfall and flood levels on May 31. Much of the information comes from the statements collected by Special Agent J. H. Hampton of the Pennsylvania Railroad Company after the flood. The statements of eyewitnesses can be found at: http://www.nps.gov/jofl/learn/historyculture/stories.htm [Accessed 2/14/2016]. These eyewitness accounts indicate that water levels ceased to rise between 12:00 pm and 1:00 pm, and may even have started to drop in both the South Fork drainage and the main stem of the Little Conemaugh River. Flood stage in the headwaters of the main stem of the Little Conemaugh began sometime before dawn, probably around 4:30 a.m., on May 31. About 10:00 a.m., maximum flood stage was reached in the Lilly area, ten miles from the South Fork dam. Further downstream in Wilmore, Portage, and Summerhill high stage occurred between late morning and 12:00 p.m. and the river levels began to fall in the early afternoon. At the town of South Fork, the Little Conemaugh began rising rapidly sometime before 9:00 a.m. and appears to have reached maximum flood stage between 12:00 p.m. and 1:00 p.m. Several witnesses (Allshouse and Brady) observed that the river never exceeded bank-full stage until the flood wave struck, and shortly before the dam failed the river level was observed to be "on a stand still" (Brantlinger).

Above the South Fork dam, tributaries in the South Fork drainage basin experienced high stage around 11:00 a.m. In the town of South Fork, at the confluence of the South Fork of the Little Conemaugh and the main stem, the South Fork rose rapidly at about 10:00 a.m. and maximum stage was reached between "shortly before noon" to perhaps 1:00 p.m., before falling slightly after 1:00 p.m., but there was overbank flooding. One observer was C. P. Dougherty, a 25-year veteran of the Pennsylvania Railroad Co. who made observations at South Fork. He was asked whether or not the stream continued to rise from 1:00 p.m. onward (Hampton, 1889). Dougherty replied: "I considered it falling a little. I was watching it when I had time, and I considered both streams were lowering a little which renewed a hope that it was also lowering at the dam, but the fall was hardly noticeable." Mr. Dougherty's statement reveals the time when the flood wave struck the town of South Fork. He reported that the clock at the railroad station stopped at 3:08 p.m., at which time the clock was thrown "out of plumb" because the floodwater floated the station off its foundation.

Downstream on the Little Conemaugh River at Conemaugh borough the water level was observed to rise until about noon and then came to a standstill, or perhaps

fell slightly, in the early afternoon. Although some railroad tracks were washed out downstream, in general the river remained at about bank-full conditions in the early afternoon until the arrival of the flood wave indicating that the runoff from the combined watersheds was also about constant during the early afternoon of May 31.

The above observations are consistent with our analysis of the time of concentration and time to peak in the South Fork watershed. These various reports indicate that for about two hours before the dam failed the flow into the reservoir was fairly constant, or even decreasing. They are generally consistent with written comments by J. Parke about conditions at the reservoir (Francis et al., 1891, p. 448–451). On the evening of May 30 he retired shortly after 9 p.m. and was awakened once toward morning by the sound of heavy rain [we note this comment is not consistent with Parke's statement 4 days after the flood, when he said: "It rained very hard Thursday night I am told, for I slept too soundly myself to hear it . . . " (Pittsburgh Commercial Gazette, 1889)]. Shortly after 6:30 a.m. on May 31 he went outside and estimated that the lake had risen roughly two feet over night. In the time it took him to eat breakfast and return to the lake he found the lake had risen appreciably, " . . . probably 4 or 5 inches . . . ." He then traveled by boat to the head of the lake where two channels entered, the South Fork itself and what is now known as Laurel Run. Parke reported that he saw the two streams at the head of the lake " . . . pouring into the lake with such an unusual roar." These observations point to a major rainfall during the night of May 30–31, with rapidly rising streams. Most eyewitness reports agree there was an extremely heavy rainfall that night, followed by rain of varying degrees through the morning of May 31. When Parke returned to the clubhouse he found the lake had risen at a " . . . wonderful rate . . . ," but gave no estimate. He was informed the water was nearly over the dam, and this is when workmen used a plow to raise a furrow to slightly elevate the central portion of the dam crest. Parke later mentions that it had been raining " . . . most all of the morning" but never referred to heavy or intense rain during this period.

Based on our analysis of this watershed, the time of concentration for runoff from a major rain event should have occurred within ∼3.6 to 7.3 h, and the time to peak Q in <7.3 h. By 6:30 a.m. the lake had already risen significantly, therefore the heaviest rainfall likely lasted for at least several pre-dawn hours, but the actual time and duration of the rainfall is not known. Russell (1889) does report that the river gage at Johnstown rose 13 ft (4.0 m) in the 24 h prior to 7:44 a.m. on May 31, and another 6 ft in the next two h. Presuming that the most intense rainfall diminished by 6 a.m. or earlier, the time of concentration and time to peak discharge for the South Fork basin should have occurred between 9:30 a.m. and 1:00 p.m. The observations summarized above show that, before dam failure, streams in the area reached their highest levels between 10:00 a.m. and 1:00 p.m., depending on

location. Therefore, if the South Fork dam could have avoided overtopping for a longer period it could readily have survived. It is a significant observation that, in the actual event, the poorly reconstructed dam underwent more than *3 h of overtopping* before failure. As shown in the calculations below, had the dam been repaired to its original specifications, overtopping would have been prevented.

4. Calculations

The following calculations examine the hypothetical scenario described by Francis et al. (1891) in which they assert that the South Fork dam would have failed even if it had been rebuilt as originally constructed in the 1850's.

4.1. Pipe flows

If the pipes and culvert had been rebuilt as in the original design they might have been used periodically to lower the lake level to permit repairs to the upstream face of the dam and would certainly have provided useful service in helping to offset the rapid rise of the lake on May 31, 1889. The fact that the sluice pipes were not replaced by the SFFHC shows that the individuals in charge of repairing the washed-out center section of the dam did not understand basic principles of reservoir maintenance. Flow to the pipes was originally controlled from a wooden tower in the lake near the center of the dam. There were five cast-iron pipe conduits each consisting of 12 individual 7 ft (2 m) long segments, which were joined together at bell socket joints. The effective length of each pipe segment was 6 ft 5.5 in (Unrau, 1980, Appx. K) and the total length was about ~23.6 m (77.4 ft). The cast-iron pipes were specified to have an "in the bore" [inside] diameter of 24 in (0.61 m) (Unrau, 1980).

By means of Darcy's 1857 modification of the Chezy formula, Francis et al. (1891) estimated the combined discharge capacity of the 5 pipes, using a head of 70 ft and pipe length of 100 ft to be 543 cfs (~15 m^3 s^{-1}). It is unclear why they chose a length of 100 ft when in their own report, on page 444, they quote W. E. Morris as saying that the pipes were " . . . about 80 feet long."

We apply an analytical equation for the inflow capacity of a pipe spillway at full flow (Chow, 1964), where

$$q = a\sqrt{\frac{2gH}{1 + K_e + K_b + K_pL}} \qquad (4)$$

q = discharge rate (cfs) (1 m^3 s^{-1} = 35.3 cfs)

a = cross-sectional area of pipe (ft^2) (3.14 ft^2)

g = surface gravity (32.2 ft s^{-2})

H = total head (ft)

K_e = coefficient for entrance loss [0.5 when pipes are mounted flush and do not protrude into upstream water column]

K_b = coefficient for bend loss (= 0 for this case)

K_p = coefficient for pipe friction loss

L = length of pipe (ft).

Given a cross-sectional flow area of 3.14 ft^2, a coefficient for pipe friction loss of ~0.01 (appropriate for cast iron), an entrance loss coefficient of 0.50, a total head of 70 ft (with the dam near overtopping and the pipe intakes several feet above lake bottom), and a horizontal pipe length of 77.4 ft, we estimate each pipe could have transmitted ~4 m^3 s^{-1}. Together the five pipes could have transmitted up to ~20 m^3 s^{-1}. This would have been the maximum discharge rate for the dam as originally built, representing flow through unobstructed pipes. If the pipes protruded into the water column on the upstream end the value of K_e would increase, up to ~0.78, and this would reduce the q value. However, diagrams of the control tower base, pipes, and culvert (Unrau, 1980, p. 48) show an upstream flush mounting ($K_e = 0.5$) with the downstream ends protruding into the culvert. Some flow loss may have occurred depending on the geometry of the flow control device at the upstream side of the pipes. Much of the difference between our flow calculation and that of Francis et al. (1891) was due to their use of an incorrect pipe length of 30.5 m.

The presence of debris on the screen leading to the pipe intakes could have caused additional flow losses. However, the intakes were at depth, which lessened the chances for clogging the grates. The analytical solution assumes that the downstream ends of the pipes discharged into an air-filled space, resulting in zero exit losses. Plate LI of Francis et al. (1891) confirms that this air-filled space at the upstream end of the culvert was ~4.3 m wide, narrowing to about 2.4 m. Historic construction drawings suggest the pipes were laid such that their bottoms were ~0.3–0.6 m above the culvert floor. If so, during pipe discharges the culvert volume would have been at least partly filled with water, which would have caused small exit losses and slightly reduced the discharge estimate given above. For example, if the pipe ends in the culvert were fully submerged (~0.6 m), the H value for Eq. (4) would be the lake depth minus 0.6 m, which would reduce the total pipe Q at the time of embankment overtopping by <2%. We have not been able to ascertain the energy slope on the floor of the culvert to determine whether it could have prevented significant filling of the culvert chamber, but diagrams shown by Francis et al. (1891, plate XLVIII) suggest it was virtually level so some submergence was likely. Another factor that could have partially filled the culvert would be the spawning of a standing wave where the flows discharged from the

pipes into the culvert, transitioning from supercritical to subcritical flow. However, as shown above, the reduction in Q from submergence effects would have been small.

Hydrodynamic erosion within the pipes and culvert was a probable cause of the South Fork dam breach in 1862. The Club's substandard repair of this earlier dam breach sowed the seeds of its eventual destruction. A review of that earlier dam breach and its possible causes is provided in the file of additional online material associated with this paper.

4.2. Auxiliary spillway

Frank (1988) presents evidence that the builders of the South Fork dam followed the original design specifications requiring that the total width of the spillways be at least 46 m and had constructed a shallow auxiliary spillway on the western side of the dam to augment the main spillway. Frank asserted that, had the crest not been lowered and kept *as originally built*, this emergency spillway would have been more than 1.1 m deep and " . . . wide enough [>70 ft (21 m)] to have carried off the waters of a storm greater than the one of 1889." Frank also concluded that the dam would not have been overtopped, but unfortunately did not provide supporting discharge calculations.

If the dam's crest had been preserved at its original height then the auxiliary spillway would indeed have functioned at high lake stages. The dimensions of this feature can no longer be measured accurately because a parking lot had been built on the western abutment. However, the 1889 topographic survey of the western dam remnant and abutment area (Fig. 7) is reasonably consistent with the dimensions given by Frank (1988), although we interpret a flow depth of 0.9 m rather than 1.1 m. To estimate flow through the emergency spillway we apply the analytical equation for long, broad-crested weirs (Chow, 1964, p. 15–33; Dingman, 1984). Analysis of these weirs is straightforward because the estimated discharges are not very sensitive to the slopes upstream or downstream from the spillway, so long as the flow depth is small compared to the weir length (d/L < 0.4) (Dingman, 1984, p. 230; Tracy, 1957).

$$Q = CL\sqrt{g}d^{1.5} \tag{5}$$

where Q = discharge [m³·s⁻¹]

C = dimensionless weir coefficient [~0.465 for long or normal weirs, where weir head divided by downstream length of weir crest <0.4]

g = gravitational acceleration [9.8 m·s⁻²]

L = length of weir crest, perpendicular to flow [21 m]

d = depth of flow over weir [0.9 m]

We estimate a discharge rate (Q) of 26 m^3 s^{-1} for the auxiliary spillway when the lake level would have been close to overtopping the *original* dam.

4.3. Main spillway and combined discharge

We used Eq. (5) to analyze the flow capacity of the main spillway for two cases: flow depth and discharge for the reconstructed dam, and a hypothetical evaluation of the same parameters for the dam at its original height. In both cases a flow width of 21 m as measured 1 m above the spillway floor was used, based on historical information (Francis et al., 1891, p. 455). Our own onsite measurement shows the present width at the narrow portion is ~1.5 m wider at a height of 1 m above the spillway floor, probably due to 125 yr of erosion of the western margin of the spillway. The eastern margin is a robust highwall, exposing sandstone beds of the Conemaugh Group. Referring to the 1889 elevations shown in Fig. 6, the main spillway for the reconstructed dam had a flow depth of ~2.0 m (1609.5 ft from Davis (1889) minus 1602.8 ft). Using Eq. (5) we calculate a discharge of ~86 m^3 s^{-1} for the main spillway of the reconstructed dam, given d = 2 m and L = 21 m.

Francis et al. (1891) estimated the discharge in the main spillway using three methods. Their preferred method gave a discharge rate of 3700 cfs (~105 m^3 s^{-1}) but used an average flow depth in the spillway of 7.5 ft (2.3 m) (Francis et al., 1891, p. 452). This flow depth corresponds to a water surface in the spillway at 1610.3 ft (1602.8 + 7.5) (see Fig. 6), but it is too high because it would have resulted in most of the plowed furrow on the eastern crest remnant being washed away. It is also almost a foot higher than the spillway water marks reported by Davis (1889). Using a more realistic flow depth of 2.0 m, their methods would yield a range of discharges from 74 to 136 m^3 s^{-1}.

Two things could have caused the discharge rates to be somewhat less than the above estimates. First, Plate XLVI of the ASCE report shows that the bed of the spillway was very rough after the flood, and this would have caused energy losses. Any roughness of the original spillway floor would have been enhanced by erosion during the flood flows. Some additional energy loss would have resulted from the fact that the main spillway is curved. The discharge rates in both scenarios (original vs. reconstructed dam) would be affected by these factors, and thus no explicit corrections were needed to make comparisons. Other factors such as the effects of flow convergence at the entries to both the main and auxiliary spillways would have been minimal due to the curved margins. Also, the spillways would not have been subject to submergence effects at their downstream ends because both spillways have long crests and terminate at steep slopes.

The cross-sectional area of the spillway at the bridge was much greater than at the narrowest part of spillway, less than 200 ft downstream. This led Francis et al. and J. Parke to conclude that the obstructions did not significantly impede the outflow, based on the assumption that the narrowest part of the spillway controlled the discharge rate. Therefore, for comparison purposes, we also evaluate the spillway flow without consideration of these obstructions.

Prior to overtopping the discharge from the reconstructed dam occurred primarily through the main spillway. A small amount of flow occurred through the newly dug ditch at the western end of the dam. In an interview, Parke noted the presence of about three feet of rock through which it was possible to cut before striking bedrock (Pittsburgh Commercial Gazette, 1889). However, in his formal letter to the committee (Francis et al., 1891, p. 449–450), Parke states that the ditch was dug 2 ft wide and 14 in deep into original ground (abutment) about 25 ft from the constructed part of the dam breast. He estimated an eventual flow depth in the ditch of ~20 in. More than three feet of excavation would be consistent with the 1889 ASCE survey data in Fig. 7. But P. Brendlinger did not see clear evidence of this ditch a year after the dam breach (Francis et al., 1891, p. 464).

Had the dam been rebuilt to its original height the maximum spillway flow depth at overtopping would have been ~3 m (see Fig. 6). This is the 10 ft freeboard for the spillway from the original dam specifications. Compared to the overtopping lake level, we allow for a somewhat lower flow depth in the spillway due to critical flow over the spillway crest, i.e., ~2.9 m. Applying Eq. (5) to estimate main spillway discharge for the dam as originally built gives a result of ~151 m^3 s^{-1}. This discharge rate is 76% greater than our estimate for the main spillway of the reconstructed dam (i.e., 86 m^3 s^{-1}s). Calculation results for the *added* discharge capacity of the dam had it been rebuilt to the higher original design are given in Table 3.

4.4. Inflow to the lake

The only actual measurement concerning the rate of rise just prior to overtopping was provided by J. Parke. In a newspaper interview several days after the flood he stated that the lake water was rising " . . . at the rate of about ten inches an hour"

Table 3. Additional discharge capacity of the South Fork dam as originally built.

Auxiliary spillway	~26 m^3 s^{-1}
Five discharge pipes	~20 m^3 s^{-1}
Added discharge for main spillway	~65 m^3 s^{-1} (151 minus 86 m^3 s^{-1})
Total *added* discharge	~111 m^3 s^{-1} (3918) cfs)

(Pittsburgh Commercial Gazette, 1889). In the same article he noted when he awoke that morning (∼6:30 a.m.; Francis et al., 1891, p. 448) he found the lake had risen " . . . until it was only four feet below the top of the dam." That suggests an *average* lake rise over five hours of ∼9½ in hr^{-1} from early morning until the dam overtopped. Parke gave a more precise rate of rise in the formal letter he wrote to the ASCE committee on August 22, 1889 (Francis et al., 1891, p. 448–451). He wrote that " . . . the lake in the hour had risen 9 inches." That observation was probably made between 11:00 and 11:30 a.m., just before the lake began overtopping the dam. We cannot know the true reliability of this observation, but it is consistent with the rate of rise through the morning prior to overtopping. Parke's estimate is the only available information that can be used to estimate the inflow rate to the lake at a specific time. No gaging data existed for the streams above Lake Conemaugh. The fact that in his letter Parke gave the value 9 inches rather than a rounded figure like "about a foot" suggests that some care was used to obtain the estimate. It should be noted that the rate of rise would have been less than 9 inches per hour if five large drainage pipes had been reinstalled beneath the dam when the SFFHC repaired the dam. We address uncertainty in Parke's lake rise estimate by including an additional 15 m^3 s^{-1} in our estimate of excess inflow to the lake. Also, our lake rise calculations for the dam *as originally designed* make the extreme assumption that high inflows continued unabated through the day, even though streams had reportedly ceased to rise.

Using our volume elevation curve for the lake basin we calculate how much excess inflow to the lake, i.e., beyond spillway capacity, would have been needed to produce this 9 in (23 cm) rate of rise. At the time of the observation the rebuilt dam was close to overtopping and the lake surface elevation would have been less than 492.5 m (NAVD 88). At this lake stage a 23 cm rise equates to an added lake volume of ∼4.14 × 10^5 m^3 or ∼115 m^3 s^{-1} as excess inflow to the lake beyond the capacity of the main spillway. According to J. Parke most of the 9" rise occurred before flow began in the emergency ditch (Francis et al., 1891, p. 449). The excess inflow of ∼115 m^3 s^{-1} is only slightly more than the *added* capacity of 111 m^3 s^{-1} that would have been available for the dam as originally built (Table 3). Therefore, if Parke's estimate was approximately correct and close to the time of peak inflow, the design of the original dam would certainly have averted overtopping long enough for the stream inflow to diminish. For our calculations of lake rise we assume some uncertainty in Parke's estimate and therefore use a larger value of ∼130 m^3 s^{-1} as excess inflow to the lake. Our calculations will show how overtopping would have been greatly delayed by the greater discharge capacity that would have been available for the original dam. Therefore, even if the inflow inferred from Parke's rate of rise had been maintained for *many* hours, the lake would not have overtopped the embankment as originally designed. As previously indicated, there is evidence from local observers that in fact the streams had ceased

to rise. Parke himself states that at about noon the lake level was " . . . almost at a stand . . . " due to the added outflow from overtopping. And in an interview given just four days after the flood, Parke stated that when he walked over the dam at about 1 p.m. (1.5 h after overtopping began), there were only about 3 inches (8 cm) of water going over the dam (Pittsburgh Commercial Gazette, 1889).

It was challenging to determine with reasonable accuracy the depth of water going over the crest of the dam because most observers, including Parke, did not specify where along the embankment that depth was observed, although one can presume it was in the center, nor do they always specify the time. Most estimates of water depth by observers are for a foot or less. For example, the ASCE report (Francis et al., 1891, p. 454) estimates that water depth over the dam was 1 ft deep for a length of 100 ft (30.5 m), and about 9 in deep for a length of 300 ft just prior to the break. Parke does state that the water was overflowing about 300 ft of the crest, but this was at about 11:30 a.m., nowhere near the time of the failure. The scenario of about 400 ft of overflow just prior to failure also seems improbable because it implies that the dam was broadly cut down from the top by water flowing over it (the breach width was ~420 ft) and there is credible evidence that this was not the case. There is no evidence in Parke's statements of broad, substantial downcutting of the embankment crest, instead he observed that a narrow "hole" about 10 ft wide and 4 ft deep was eventually cut into the outer face. Washovers through this trough severely eroded the downstream face and toe of the dam, laterally thinning the embankment until "the pressure of the water broke through" (Francis et al., 1891, p.451) creating a rapidly growing trough in the dam breast. Flow through this breach was so great that Parke observed a ≥10 ft drawdown in the lake surface that extended about 150 ft back from the crest (p. 451). The highest overtopping estimate of 16 in was given by A. Y. Lee (Unrau, 1980, p. 134) as the depth when the dam failed, but it is not clear on what he based his conclusion. However, if Schwartzentruver's statement (Russell, 1964) that he and his friends crossed the dam on foot shortly before failure is indeed accurate then it is highly improbable that water depth on the crest could have been up to 16 in, or even 1 ft on the central portion. Parke's estimate of perhaps 3 in is much more likely. At the very least this substantiates his conclusion that the water level was virtually at a stand. Therefore, it appears that the stream inflow had peaked around noon, which would be consistent with our assessment of the time of concentration and time to peak for the South Fork of the Little Conemaugh River. Accordingly, the extreme rate of rise reported by J. Parke for late morning was not sustained and probably represents the general time of peak inflow to the reservoir.

Fig. 12 shows the discharge capacity of the dam via its two spillways and five sluice pipes, assuming the dam had been rebuilt to its original specifications. Table 4 summarizes key data for the dam and watershed. Our analysis shows that the lake surface at time of dam failure had an elevation of ~492.5 m (NAVD 88).
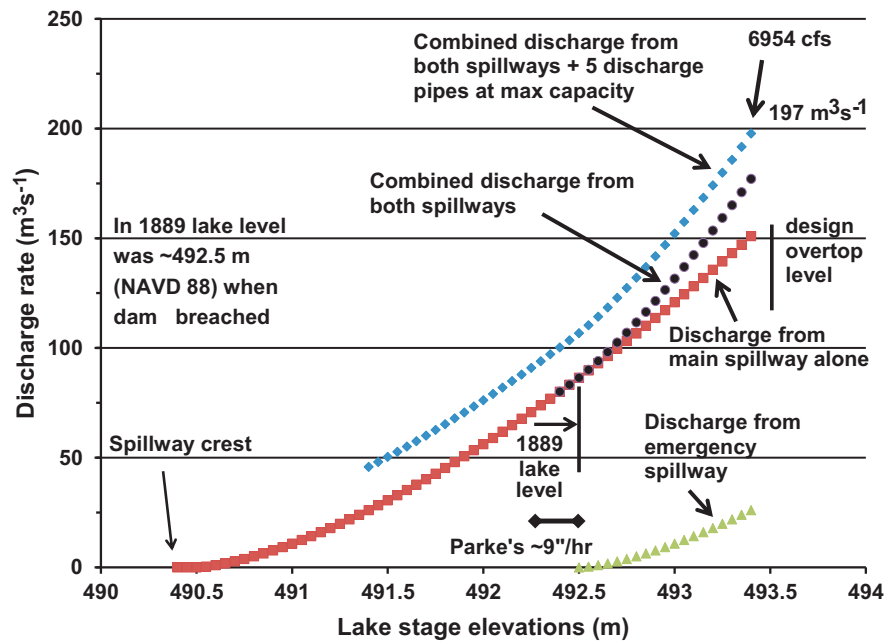
Fig. 12. Discharge capacity for the South Fork dam as originally designed, with crest ~1 m higher. The base level of this chart is ~490.4 m which is our estimate of the modern-day elevation of the subsoil spillway crest (see Section 2.3.2). Overtopping would have occurred at ~493.5 m (NAVD 88) if the dam had been rebuilt to its original crest height. The spillway flow level would have been ~0.1 m less.

The dam as originally built could have discharged floodwater at a maximum rate of ~197 m^3 s^{-1} (6954 cfs) given a spillway flow surface elevation of 493.4 m (NAVD 88), more than 2 times greater than the estimated discharge capacity of the reconstructed dam. This greater discharge capacity has major implications for the survivability of the dam *as originally built*.

4.5. Inflow calculations by the investigation committee (Francis et al., 1891)

Part of our retrospective analysis of the committee's conclusions is an evaluation of their lake inflow calculations, which led them to believe the dam would have been overtopped and destroyed even if it had been rebuilt to its original design. Francis et al. (1891, p. 452–453) give three estimates of the rate of rise of the lake stage, and present the calculations as evidence that the rate of lake inflow continued to increase until the dam failed. These are shown in Fig. 13. Their first estimate was based on a rise in lake level of 10 in/h around 10 a.m. Assuming a spillway flow depth of 5 ft, they estimated the total inflow to the lake at 7208 cfs (204 m^3 s^{-1}). No reference was cited for the source of the 10 inch rise per hour or for the reported 10 a.m. timing. We found Unger's testimony for the Pennsylvania Railroad where he stated that the lake was rising "at the rate of about ten inches an hour" at about 6 a.m. At 6:30 a.m. John Parke (Francis et al., 1891, p. 448) noticed

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Table 4. Summary of key data for the South Fork dam and watershed.

| Dam comparisons	Original dam	Rebuilt dam
Embankment height	72 ft (22 m)	≤70 ft (~68–69 ft in center)
Overtopping elevation (NAVD 88)	~493.5 m	~492.5 m (~1616 ft); lake level at time of dam breach
Discharge capacity (Q) at overtopping		
Main spillway	~151 m^3 s^{-1}	~86 m^3 s^{-1}
Auxiliary spillway	~26 m^3 s^{-1}	ditch ~ < 10 m^3 s^{-1}
Five discharge pipes	~20 m^3 s^{-1}	0 m^3 s^{-1}
Total Q	~197 m^3 s^{-1} (6954 cfs)	~96 m^3 s^{-1} (3430 cfs)
Estimated time of embankment overtopping	Would not have overtopped	~11:30 a.m. (actual)
Spillway floor elevation (NAVD 88)	~490.4 m (crest)	~490.4 m (crest)
Hydraulic and basin data		
Basin drainage area above Lake Conemaugh	~53 mi^2 (137 km^2) (Brua, 1978)	
Time of concentration (t_c), South Fork of the Little Conemaugh R.	3.6 to 7.3 h	
Time to peak inflow to lake after period of most intense rainfall	2.5 to 5.1 h (≤7.3 h in extreme events)	
Rate of lake rise ~10:30–11:30 a.m. just before initial overtopping of dam	~9 in h^{-1} (J. Parke; Francis et al., 1891, p. 449) [corresponds to lake inflow of 115 m^3 s^{-1} beyond main spillway capacity of rebuilt dam]	
Estimated peak lake inflow before 11:30 a.m.	201 m^3 s^{-1} [spillway (86) + excess inflow (115)] [inflow of 216 m^3 s^{-1} used in our analysis to reduce chance of underestimating inflow]	
Volume of lake at time of 1889 dam breach	1.455 × 10^7 m^3 (below stage 492.56 m, NAVD 88)	
Time to drain lake to base of the upper breach	≥65 min	
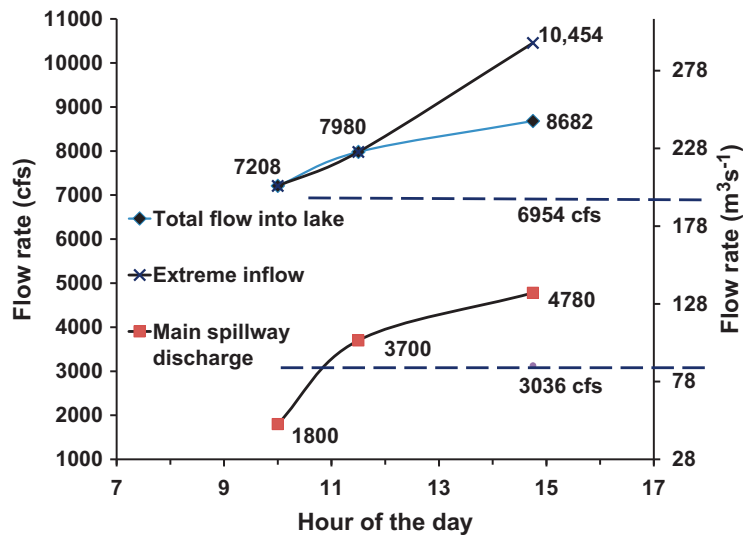
Fig. 13. Various inflow rates estimated by Francis et al. (1891) for the South Fork dam and reservoir. Upper two plots represent total estimated inflows to reservoir. Lower plot represents estimated discharges in main spillway. Lower blue dashed line is maximum discharge in main spillway based on 6.7 ft (2.0 m) height of flood marks measured by Davis (1889). We estimate 86 m³ s⁻¹ for the spillway at that time. Upper dashed line is our estimate of the total discharge capacity of the dam as originally built (197 m³ s⁻¹).

that the lake had risen about 2 feet during the night, went to get breakfast and after eating returned to the lake and found that it had risen 4 or 5 inches during breakfast. Perhaps the ASCE members estimated ½ h for Parke's breakfast and ~10 in h⁻¹ rise, or they saw the newspaper account of his early morning estimate of about 10 in h⁻¹ rise (Pittsburgh Commercial Gazette, 1889). However, these lake rise estimates were much earlier than 10:00 a.m.

Francis et al. (1891, p. 452) based another total inflow estimate of 7980 cfs (226 m³ s⁻¹) on J. Parke's reported 9 inch per hour rise from about 10:30 to 11:30 a.m., based on a spillway flow depth of 7.5 ft which, as previously discussed, is improbable based on topographic data. Using a spillway flow depth of 6.7 ft, which is more consistent with observations, yields a lower inflow rate. Our LiDAR-based volume elevation curve, along with Parke's estimated rise, and hydraulic analysis of the main spillway indicates a total lake inflow of ~197 m³ s⁻¹ near the time of overtopping in 1889. Francis et al. (1891) also evaluated a scenario around 10 a.m. with a spillway depth of 5 ft, with the lake level reportedly rising at 10 in h⁻¹. However, at this rate, in the 90 min from 10 a.m. until 11:30 a.m. the water level could not have risen 2.5 ft (7.5 − 5 ft). It would only have risen ~15 in, and probably less after taking into account the nonlinearity of the volume elevation curve (Fig. 9).

We also examined the third and most critical calculation of inflow to the reservoir by Francis et al. (1891). They estimated a water rise of 6 in per hour *in the hour before the dam breach* (i.e., the hour before 2:55 p.m.), implying an excess lake inflow of about 2911 cfs. The dam crest was being actively overtopped at that time. They calculated that the main spillway was discharging at a rate of 4780 cfs using a flow depth of 8.7 ft. Overtopping flow over the dam crest was estimated at 991 cfs, resulting in a combined lake inflow rate of 8682 cfs (247 m^3 s^{-1}). As further support, the committee cited Parke (Francis et al., 1891, p. 451) who stated that after the dam failed " . . . there still remained in the bed of it a violent mountain stream 4 or 5 feet deep, with a swift current . . . [that] showed no signs of diminishing in volume until late the following day. . . . " However, a spillway flow depth of 8.7 ft was implausible. Fig. 6 shows that the spillway flow in this scenario was greatly overestimated. The assumed flow depth of 8.7 ft in the spillway is much greater than the observed 6.7 ft (Davis) and would also have required unrealistically deep overtopping levels at the dam crest that would have obliterated all evidence of the raised furrow on the eastern dam remnant. That furrow was preserved, and therefore the third inflow estimate by the committee cannot be correct. The entire western dam remnant would also have been submerged. If the spillway had been flowing with a depth of 8.7 ft, the lake stage would have risen to ≥1611.5 ft (1889 reference frame) and the entire dam would have been overtopped to an average depth of >1.5 ft based on the survey measurements the committee themselves sponsored (Fig. 6). Even the highest points on the western dam remnant would have been overtopped by ~0.3 ft. If we consider only the top width of the breach (420 ft), hydraulic calculations show that a mean flow depth of >1.5 ft would yield an unrealistic discharge of >1760 cfs (>50 m^3 s^{-1}) just for the overtopping flow, not the 990 cfs estimated by the committee. Clearly the entire dam was never overtopped, and there is no evidence that the lake stage rose half a foot (0.15 m) in the final hour before the dam breached, that is, after ~1:45 p.m. As noted earlier, Parke reported that after overtopping the lake was virtually at a stand due to the added discharge over the dam crest. The committee's conclusion that the lake was continuing to rise appears to be implausible.

Interestingly, the committee did a further calculation (Fig. 13) to estimate an upper limit for the inflow that day, by assuming a rainfall rate of 2/3 inch (1.7 cm) per hour on the watershed (20,900 cfs) and that half of this influx (10,450 cfs) would discharge into the lake (Francis et al., 1891, p. 454). The committee predicted that a flow rate of this magnitude or higher could have entered the reservoir by 4 p.m. (or later) on May 31, more than an hour after the actual breach. This calculation appears intended to show that water levels could only have been expected to go higher, strengthening the basis to conclude the dam would have failed even if it had been repaired to its original specifications. However, the committee did not

consider what such an influx would mean with respect to spillway flow depths and crest overtopping levels, which likewise render this extreme influx scenario to be implausible. It is a combination of observations that allow us to reject the magnitude and timing of these extreme influx scenarios: (1) upper flood stage marks in the main spillway (Davis, 1889), (2) preservation of furrow mounds on the eastern dam remnant (Wellington and Burt, 1889), (3) observations that the stream stages had peaked or even begun to drop, and (4) our assessment of a plausible time of concentration and time to peak for the watershed that is consistent with the stream level observations. The fact that both the South Fork and Little Conemaugh had ceased to rise or had begun to recede about 2 h before the dam breach was apparently not known or recognized by the committee. And finally, using our breach flow calculations, the committee's extreme inflow estimate ($\sim$300 m^3 s^{-1}) would have sustained a flow depth in the inner breach of $\sim$5.7 m after the lake had drained. This flow depth far exceeds the observation by Parke (Francis et al., 1891, p. 451) that it was " . . . a stream 4 or 5 feet deep . . . " Therefore the committee's extreme inflow estimate is inconsistent with our calculations and 1889 observations.

4.6. Estimate of peak flood discharge rate

The center of the South Fork dam failed catastrophically more than 3 h after overtopping began. Dam breach methods have been used by other authors to estimate the peak discharge. The scientific literature contains several estimates of peak discharge rate for the Johnstown Flood of 1889. MacDonald and Langridge-Monopolis (1984) give a range of 5600 to 8500 m^3 s^{-1}, but their source is Pagan (1974), who gives the same range but no reference or method of calculation. Froehlich (1995) presents estimated peak outflows from a series of breached embankment dams, including the South Fork dam. He also gives a peak discharge rate of 8500 m^3 s^{-1} and indicates the estimate is based on reservoir volume change over a 30 min period. Froehlich (1995) identifies the rebuilt South Fork dam as having "homogeneous earthfill," which is partly correct. As designed the embankment was a zoned earthfill dam with puddle layers on the upstream half. The repaired center section consisted of randomly placed fill and was not zoned in the same way. Froehlich (1995) reported a lake volume of 1.89×10^7 m^3 and a height of water above the breach bottom of 24.6 m. This latter number is not correct as the depth was $\sim$21.3 m, and our LiDAR-based volume is 1.455×10^7 m^3. Singh (1996) reports a lake volume of 1.90×10^7 m^3 and flow duration of 0.75 h. In modeled hydrographs Singh (1996, Fig. 6.4) shows peak discharges>7000 m^3 s^{-1}. Pierce et al. (2010) give a peak outflow of 8500 m^3 s^{-1}, citing Wahl (1998), but they also use the excessively large water depth of 24.6 m that had been listed by Wahl (1998).

The breach in the South Fork dam is shown in Fig. 14. The maximum possible discharge rate for the flood would have occurred if the entire breach formed suddenly with the lake stage at maximum overflow depth. To evaluate the reported peak discharges we applied equation (11b) of Walder and O'Connor (1997) to estimate peak discharge through the breach,

$$Q_o = \left[c_1 r(D_c - b_f) + c_2 h \cdot \cot\theta\right] g^{1/2} h^{3/2} \qquad (6)$$

Where Q_o = discharge from lake (m^3 s^{-1})

r = bottom width of breach (m) divided by breach depth (m) (dimensionless)

D_c = height of dam relative to dam base (m)

b_f = height of breach floor relative to dam base at end of flood

θ = slope of breach sides (degrees)

g = surface gravity (9.81 m/s^2)

h = lake level relative to breach floor (m)

c_1, c_2 = numerical constants related to breach shape ($c_1 = 0.405$ and $c_2 = 0.544$); these values represent case that neglects energy loss as floodwater approaches the breach, and also assume no tail-water effects at the outlet; (Walder and O'Connor, 1997).

We used Eq. (6) to estimate the peak discharge, treating both the "inner" breach (center of the breach; see Fig. 14) and "outer" breach (breach segments outside the
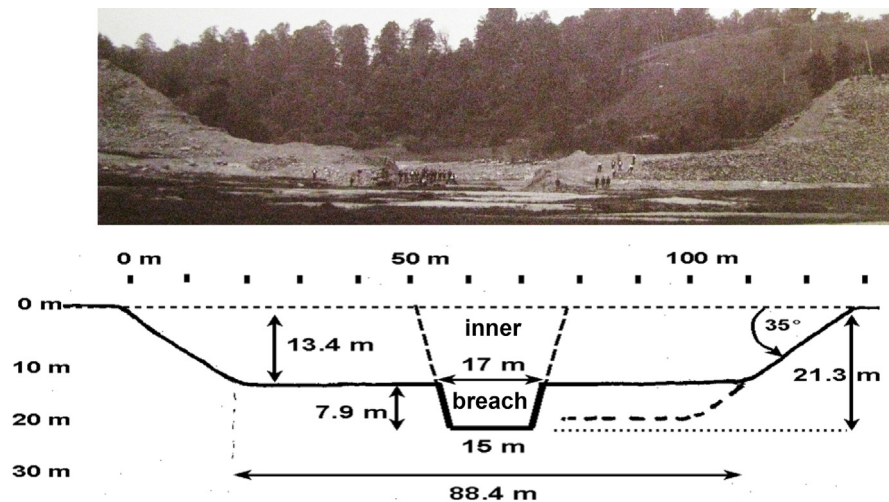

Fig. 14. Geometry of the breach in the South Fork dam. There is no vertical exaggeration. Dimensions based on post-flood dam survey data (Francis et al., 1891) and 1889 photographs. Dashed line at lower right shows where dam remnant was later removed to build a double rail line through the old breach. Note numerous people standing on the dam remnants in post-flood image at top.

17 m wide center) as separate calculations. As surveyed in 1889, the top, or maximum, total width of the breach was 129.6 m. The upper part of the breach had an average depth of ~13.4 m and a bottom width of ~88.4 m. Subtracting the mean width of the inner breach gives a width of 88.4–19 m = 69.4 m ($r = 5.18$ and $\theta = 35°$). The inner breach was cut to the base of the dam and was ~15 m wide at the base and more than 21 m high ($r = 0.70$ and $\theta = 73°$). The combined dam breach discharge of ~8870 $m^3 \cdot s^{-1}$ approximates the maximum possible discharge that could have occurred through the breach. At the moment of general dam failure, flow was also occurring through the main spillway and ditch, plus flow over the central portions of the crest. There is no explicit addition of overtopping flow because that is incorporated in the breach flood discharge. The spillway transmitted ~86 $m^3 \cdot s^{-1}$, and the ditch about 10 $m^3\ s^{-1}$, resulting in a combined flood discharge of ~8970 $m^3 \cdot s^{-1}$. We consider it reasonable to include a 15% increase in breach discharge rates to incorporate uncertainty (see Section 4.7). The result is a maximum peak discharge of 10,200 $m^3\ s^{-1}$ through the breach, plus ~100 $m^3\ s^{-1}$ flow through the spillway and ditch.

There is little doubt that the upper part of the dam breach formed very rapidly (Kaktins et al., 2013). This may not have been the case for the bottom 7.9 m of the inner breach as its erosion through the original puddle layers at the base of the dam may have been more gradual. A more realistic value for the instantaneous peak produced by the initial flood wave would incorporate rapid failure of the upper 13.4 m of the breach (7100 $m^3 \cdot s^{-1}$) and flows through the main spillway (86 $m^3 \cdot s^{-1}$) and ditch. The result is an estimated peak discharge (Q_p) of ~7200 $m^3 \cdot s^{-1}$ for the instantaneous discharge rate during catastrophic formation of the upper breach. The estimate of peak discharge given by multiple authors (8500 $m^3 \cdot s^{-1}$) is probably too high, although it is less than our estimate of the highest plausible discharge of ~10,300 $m^3 \cdot s^{-1}$.

4.7. Hydrographs of the 1889 flood

We have adapted an existing mathematical model to generate discharge and lake stage hydrographs of the dam breach flood. As shown in Fig. 14, the dam breach can be viewed as two trapezoids comprising an inner breach and two "wings" of an outer breach, which we combined. Only the smaller inner breach eroded to the base of the dam. This complex breach prevents the use of most existing analytical models that assume relatively simple, geometric shapes. To generate hydrographs we modeled the two trapezoids as independent breaches that simultaneously drained the lake, along with residual flow through the spillway during the early minutes. The breach geometry was greatly simplified at mid to late times when all flow was in the bottom of the inner breach with an initial depth of 7.9 m. To analyze this portion of the hydrographs we applied equations from Walder and

O'Connor (1997). These included their lake stage differential equation (10b) (not reproduced here) and weir equation (11b) (Eq. (6) above).

We analyzed two principal breach scenarios. In the first the entire complex breach forms fast, such that the breach fully forms before significant drawdown of the lake. This produces the highest peak discharge and drains the lake in the shortest time. In the second scenario the outer breach and upper part of the inner breach form instantly. This condition establishes the lower range of peak discharge. The zone between these scenarios encompasses all hydrographs that would result from gradual erosion of the lower 7.9 m of the inner breach. That the upper part of the complex breach did form almost instantly is supported by the eyewitness account of U. Ed Schwartzentruver, retold many years later in an interview, that trees immediately below the dam were felled by an air blast before the flood wave reached them (Russell, 1964; Kaktins et al., 2013). Uprooting of trees by wind would require speeds exceeding $\sim$33 m s^{-1} (NOAA, 2012), which is faster than the flood wave would have been. Similar wind speeds might have been possible near the leading edge of a dam breach flood wave.

Hydrographs were developed by summing the discharges from the inner and outer breaches over time as the lake gradually drains (Fig. 15). The model fully incorporates the complex breach shape in the South Fork dam and is based on the storage-elevation curve of the reservoir (Fig. 9) and a dam breach weir equation that allows specification of the breach side slopes. Eq. (6) was used with data from the storage-elevation curve by numerically evaluating lake stages and volumes that correspond with integrated model discharges through the breaches. One scenario
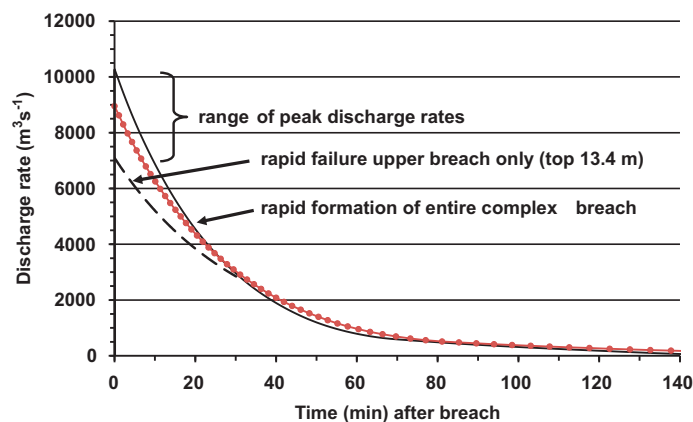

Fig. 15. Dam breach discharge hydrographs for the 1889 flood. Red dotted line represents scenario where the entire breach formed instantly. Solid black line includes 15% increase in initial discharge to incorporate uncertainty (see text). These calculations represent maximum theoretical discharge through the breach and spillway, with Q_p $\sim$8970 to 10,300 m^3 s^{-1}. Dashed black line represents early-time discharge from only the top 13.4 m of the breach. This line intersects the vertical axis at $\sim$7100 m^3 s^{-1}. Adding $\sim$100 m^3 s^{-1} for the spillway and ditch yields the minimum Q_p of $\sim$7200 m^3 s^{-1}.

we analyzed includes a 15% increase in initial breach discharge rate to incorporate uncertainty in the overall analysis. Sources of uncertainty may include complexity of flow through the unusual "double" breach, accuracy of breach reconstruction from 19th century data and images (present breach is highly altered from original), and the possibility that a transient deeper channel formed in the lower breach that was infilled during the waning flood. The latter point is less likely because the original dam base was excavated to bedrock.

We include a hydrograph plot that represents early-time (0 to 30 min) discharge through the top 13.4 m of the breach, assuming it formed instantly (dashed black line in Fig. 15). The lowest part of the "inner" breach may have formed gradually. Hydrographs for all scenarios of erosion (rapid or gradual) of the lower 7.9 m of the inner breach (see Fig. 14) would lie between the red line and the dashed black line in Fig. 15. We emphasize that all the discharge hydrographs represent conditions at the breach and for the stream immediately below the dam and upstream from the juncture of the South Fork with the Little Conemaugh River. Our present work focuses on the dam breach and does not analyze the flood routing along the Little Conemaugh to the former boroughs that now comprise Johnstown.

The lake stage hydrographs are shown in Fig. 16, illustrating the decline in lake level during the flood. Plots for three lake inflow rates are shown. Spillway flow is
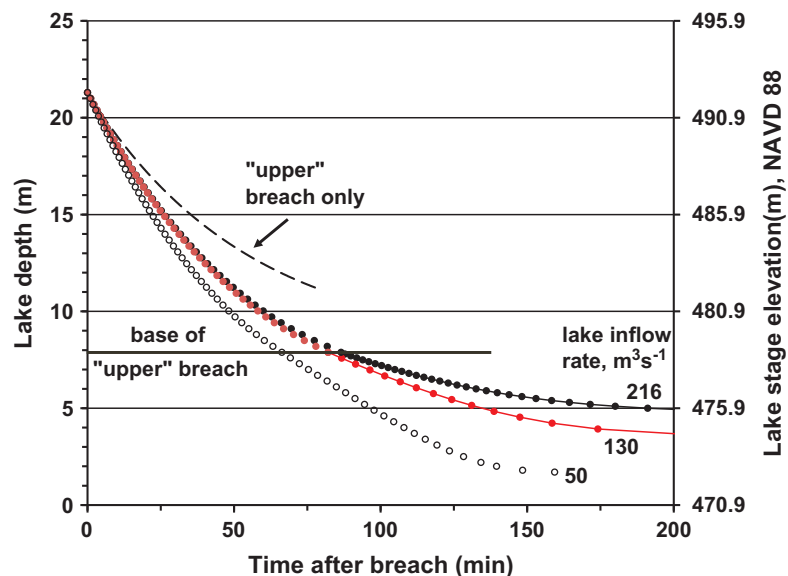

Fig. 16. Lake stage hydrographs for Johnstown Flood of 1889. Curves are plotted for three different lake inflow rates. Dotted lines represent cases where the entire complex breach formed rapidly, which would have drained the lake in the shortest time. Plot of open circles includes 15% increase in initial discharge rate to incorporate uncertainty (see text). Dashed line shows hypothetical lake stages if only the upper 13.4 m of the breach had formed. The zone between the dashed and dotted lines represents a family of curves for all plausible erosion times for the lower 7.9 m of the inner breach.

incorporated only in the early minutes before the lake fell below its crest. The plot for total inflow of 216 m^3 s^{-1} corresponds to the lake rise reported by J. Parke (9 in h^{-1}) just before overtopping, plus an added inflow of 15 m^3 s^{-1} to address uncertainty. Curves for two lesser inflows are shown because the lake inflow likely declined during the more than 3 h from the time of initial overtopping until the dam failed. At least 65 min were needed for the stage to fall below the floor of the upper breach. At late times the lake levels approach equilibrium as discharges thorough the lower breach approach the lake inflow rates. To help verify the calculations for the discharge and stage hydrographs, Fig. 17 was prepared to demonstrate the close water balance in the model. This calibration plot compares cumulative model flood discharge with the storage elevation curve for Lake Conemaugh (Fig. 9).

Some observers reported that Lake Conemaugh drained in as little as 37 min to as much as 75 min (McCullough, 1968, p. 102; Kaktins et al., 2013). J. Parke clearly stated (Francis et al., 1891, p. 451), "I do not know the actual time [to drain the lake through the breach], but it was fully forty-five minutes." Some modern compilations of dam breaches (Singh, 1996; Froehlich, 2008) cite the 45 min drain time but do not report it as a *minimum* estimate by Parke. Wahl's (1998) report transposed numbers, noting that the breach took 45 min to fully form and the lake took 3.5 h to drain. We clarify that the dam sustained overtopping for>3 h before catastrophically breaching in a very short time, then drained rapidly.

The 1889 observations along with the hydrographs give strong support for the entire breach forming quickly. *At least 65 min were needed* for the lake stage to
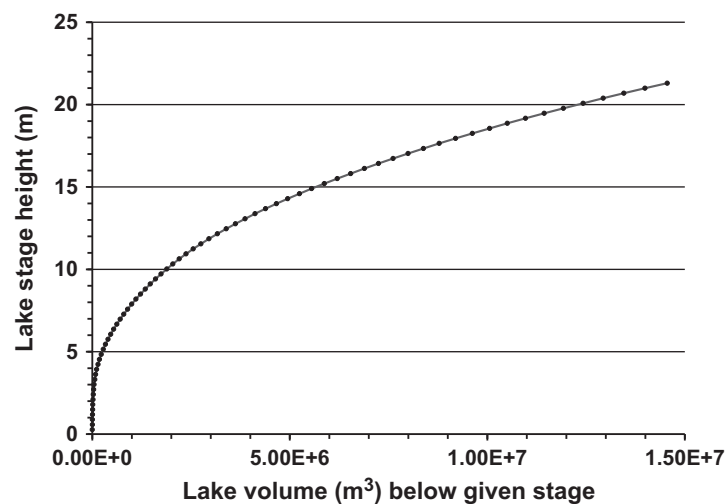

Fig. 17. Calibration plot shows good water-balance fit between the storage-elevation curve for Lake Conemaugh in 1889 (black data points) and total lake volume minus cumulative model discharge for the rapid breach scenario (curved line). For this plot model discharge does not include lake inflow. The curve match is consistent with a lake floor shape factor (*m*) of 2.7, which represents a basin with gently sloping sides and bottom (see Walder and O'Connor, 1997, p. 2341).

drop enough to expose the base of the upper breach, and parts of this base may have been exposed in a shorter time. This may have led some observers to think the lake had fully drained in that time. However, the flow depth would still have been ~8 m in the lower breach. Certainly after 65 min large areas of former lake bottom would have been exposed by the rapidly falling lake level. It is therefore easy to see why different observers gave a wide range of time to drain the lake. If the lowest part of the inner breach took a significant time to form, then the drainage time should have been longer. One further insight from the hydrographs is that tailwater effects at the outlet were probably not significant, otherwise even more time would have been needed to drain the lake.

As shown at lower right in Fig. 16, at late times the lake stage would have approached equilibrium as breach outflow dropped to near the rate of lake inflow. For example, if the inflow rate fell 40%, from 216 m^3 s^{-1} before overtopping (around 11:30 a.m., per Parke) to 130 m^3 s^{-1} several hours later, the reduced inflow could have sustained a flow depth in the lower breach>3 m. This is a greater flow depth than estimated by Parke, who commented that when the lake drained there still remained in its bed a stream 4 or 5 feet deep (Francis et al., 1891, p. 451). However, it is not clear when or how carefully his observation was made, and it may have been some hours after the breach when discharges were even less. It is also possible that the inflow rate dropped even lower than 130 m^3 s^{-1} by mid-afternoon on May 31st.

The tendency of floods to recede relatively quickly in this "flashy" watershed is well illustrated by the behavior of the flood hydrograph from the 1977 Johnstown flood, which resulted from greater rainfall amounts than the 1889 flood. As measured at the USGS gage at East Conemaugh (Fig. 11), this hydrograph on the main stem of the Little Conemaugh peaked at 1100 m^3 s^{-1} (Brua, 1978). The discharge then *fell by 50% in only a few hours*. In this extreme runoff event, the discharge rate at this gage downstream from South Fork on the Little Conemaugh exceeded 50% of its peak for only 5 h. We note there were some differences in proportionate land use between 1889 and 1977, including surface mining, which would have contributed to the watershed response rate. However, the watershed response in 1889 for the South Fork of the Little Conemaugh should have been rapid given its much smaller drainage area.

5. Discussion

We respectfully disagree with the ASCE committee that reviewed the cause of the failure of the dam (Francis et al., 1891). They had concluded that the modifications to the dam by the SFFHC " . . . cannot be deemed to be the cause of the late disaster as we find that the embankment would have been overflowed and the

breach formed if the changes had not been made." We question the committee's conclusions for five reasons:

First, if the drainage pipes had been replaced they would have been opened early on May 31st to their maximum capacity, which would have reduced the rate of rise observed by Parke. The issues discussed in the additional online material about possible hydrodynamic damage to the pipes and culvert would not have been a concern because releases from the dam were no longer needed to supply water to a canal. Therefore newly installed pipes and a repaired culvert would only have been used during floods or to lower lake levels to allow embankment repairs. Second, the committee failed to recognize that the dam crest likely had been lowered by the SFFHC more than 0.6 m, probably as much as 0.9 m, including settling of the embankment. The storage-elevation curve (Fig. 9) shows that an added 0.9 m of lake stage could have stored an additional 1.6 million m^3 of water without overtopping. Third, in the original design the main spillway would have had much greater discharge capacity and the discharge pipes and auxiliary spillway would have been functional, but the committee never acknowledged that this smaller spillway even existed. Fourth, contrary to the committee's claims of an increasing rate of rise of the lake stage until dam failure, there is evidence that flood levels in both the South Fork and the Little Conemaugh River had stabilized and begun to fall, if only slightly, by 12:00–1:00 pm, suggesting that the time of concentration and peak in the runoff hydrograph had occurred hours before the dam breach. And fifth, the poor reconstruction of the older dam breach did have several adverse consequences. Apart from the fatal lowering of the dam crest, the random-fill technique used to fill the previous breach (rather than the clay "puddling" method) was a major factor in the disaster by making it possible for a large portion of the dam's core to become saturated during high lake stands. The committee itself reported (Francis et al., 1891, p. 454) that "All the material put in [by the club] . . . to repair the breach of 1862, appears to have been washed out, together with part of the old embankment . . . " They also reported (p. 454) that exposed parts of the *original* dam showed that they " . . . offered great resistance to washing and that [the work] was originally selected and put in with the requisite care to make a sound embankment."

We prepared Fig. 18 to clearly show how the dam as originally built would have resisted overtopping on May 31, 1889. Even if extremely high lake inflows had continued unabated, overtopping of the dam at its original design height would have been averted for 14 h. In the absence of alternate failure mechanisms such as piping, the dam would have been preserved because lake inflows would have substantially diminished during the afternoon and evening. In the actual event, if the estimated peak inflow had time to fall just 9%, from ~216 to ≤197 m^3 s^{-1}, overtopping would have been entirely prevented had the dam been rebuilt to its original height. Fig. 18 also reveals that had the reconstructed dam been built only
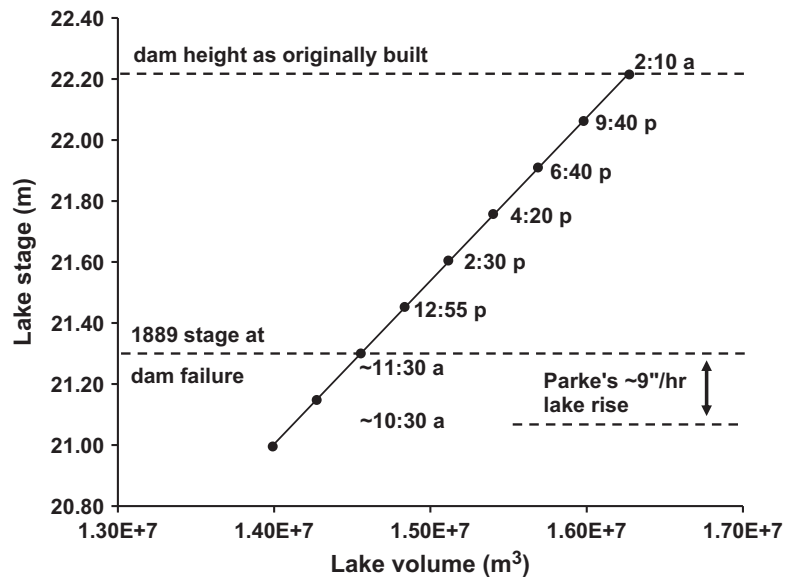
Fig. 18. Graph showing time needed to reach various hypothetical lake stages. Graph was prepared using the discharge capacity curves in Fig. 14 with the storage elevation curve in Fig. 11. The plotted times assume that a constant lake inflow of ~216 m³ s⁻¹ would continue for many hours. Even in this extreme and unrealistic scenario, the nonlinear increase in spillway and pipe flow capacities and rise in lake storage would have averted overtopping for ~14 h. In reality the dam would never have been overtopped.

0.6 m higher, overtopping would have been delayed more than 7 h even at the high and constant inflow rate assumed to develop this figure.

It is not well known that the reliability of the South Fork dam was tested just a few years after it was completed, when in the spring of 1856 the reservoir overflowed at an unspecified place after a rapid snowmelt (Unrau, 1980, p. 51). We have not found details about where the overflow occurred, but given the higher dam crest it must have flowed through the auxiliary spillway on the western abutment. With the dam at it design height the reservoir appears to have performed as intended, with two functioning spillways and the discharge pipes also probably opened. A leak in the dam was reported at that time but was soon repaired. In fact, the Pennsylvania State Engineer inspected the South Fork dam later that year. "The Western Reservoir [Lake Conemaugh] was examined and found to be in excellent condition. It furnished a sufficient supply of water to keep up the [canal] navigation when other sources had entirely failed" (Gay, 1856, p. 16). He had reported that the spring floods severely damaged other impoundments in the region, including Piper's and Raystown dams on the Juniata River. Eighty-foot breaches occurred in both of these dams.

There are useful insights to be gained from the resilience of another earthen dam in the region that was built with a "puddled" clay core. This dam was on Mill Creek

four miles from Johnstown, and it survived the May 1889 event. We know this from the minutes of the Board of Trade and Citizens Meeting, June 19, 1891, which state that the dam was 310 ft long and 25 ft high, built in 1834 with a spillway 44 ft wide and a "puddle wall" 7.5 ft wide within the center of the dam. The minutes go on to document that a "freshet" [flood] occurred on July 2, 1889 and that the spillway was insufficient to carry the flow. The *whole dam* was overtopped by this flood but it did not fail, providing an example of the resilience of the original "puddle wall" construction technique. Unfortunately, the SFFHC did not use this technique to repair the South Fork dam. Kaktins et al. (2013) argue that the fill contained plastic clays, which when wetted would have had very low shear strength. Therefore, liquefaction effects may have been largely responsible for the very rapid upper breach formation. Francis et al. (1891, p. 446) commented that hauling fill by [horse] teams over the freshly deposited material " . . . made a fairly compact embankment on the upper side of the stone embankment." But since there was a definite sag in the central, filled portion of the embankment any compaction was of a limited nature.

It is interesting to compare the committee's conclusions about the South Fork dam (Francis et al., 1891) to their earlier investigation of the Mill River dam failure in Massachusetts (Francis et al., 1874; Sharpe, 2004). James Francis and William Worthen served on both committees. The members criticized the material used to make the Mill River embankment, and that it could not be relied on to make the structure water-tight. During the construction there " . . . was no sufficient inspection, so peculiarly important in a work of this description . . . " "The remains of the dam indicate defects of workmanship of the grossest character." In the discussion section, member Worthen went on to write:

> Men were employed who were ignorant of the work to be done, and there was nothing like an inspection, although money and life depended upon it. I do not believe, however much we are an evolved species, that we are derived from beavers; a man cannot make a dam by instinct or intuition.

Neither Worthen nor Francis reflect this philosophy in their role 16 years later as investigators of the South Fork dam failure. The need for engineering inspection and water-tight embankments was equally important during the SFFHC's repair of the South Fork dam. And yet, when confronted with the many changes made to that dam and the poor methods used to repair the embankment, including lack of engineering inspection, Francis et al. (1891) concluded that the changes and repairs to the dam were not responsible for the disaster. They even went so far as to conclude that if the embankment had been rebuilt to its original height the result may have been *greater* loss of life. Clearly there were inconsistencies in philosophy between the ASCE reviews of the South Fork and Mill River dam failures.

Despite numerous lawsuits, the SFFHC was never found financially liable for losses of life or damages. One reason for this was that Benjamin Ruff, who supervised the rebuilding of the dam, had died two years before the 1889 dam breach and could not be held accountable. Most of the legal actions ended before publication of the ASCE report by Francis et al. (1891).

6. Conclusions

We calculate a LiDAR-based volume of 1.455×10^7 m^3 for Lake Conemaugh at the time of the 1889 dam breach. The peak discharge was likely in the range from 7200 to 8970 m^3 s^{-1}, with the lower part of this range being more likely because the deepest part of the complex breach may have formed gradually. Considering uncertainty, we estimate an upper limit on the peak discharge of 10,300 m^3 s^{-1}. The reservoir took more than an hour to drain, contrary to older claims that the lake drained in 45 min. If the entire complex breach formed rapidly, more than 65 min would have been needed to drain the lake to the floor of the upper breach. Part of the lake would still have been nearly 8 m deep at that time.

In analyzing topographic surveys at the site we found that the 1889 dam survey elevations are systematically 1.9 m (6.2 ft) lower than the modern GPS reference frame. Another of our findings is that soil genesis on the bare rock spillway has produced as much as half a meter of soil in>120 years following the dam breach. The average rate of soil accumulation exceeds 4 mm yr^{-1}.

Although Francis et al. (1891) stated that the mode of repairing the breach was not according to best practice, they nonetheless concluded that " . . . failure of the dam cannot be attributed to any defect in its construction. The failure was due to the flow of water over the top of the earthen embankment, caused by the insufficiency of the wasteway . . . " However, we find that the changes made to the South Fork dam were indeed responsible for the disaster, having altered its original design and rendered it highly vulnerable to overtopping. The dam could indeed have survived the rainfall event of May 30–31, 1889 had it been maintained as built in 1853 with a higher crest, a functioning second spillway, five drainage pipes, proper well-compacted fill with puddle layers, riprap replacement of proper size, and no bridge or fish screens across the main spillway. The discharge capacity of the original dam was more than *twice* that of the reconstructed dam. The dam as originally designed could have averted overtopping for as much as 14 h even under extreme conditions of inflow duration and rate. Such extreme conditions did not exist because observations of local streams indicate that flows into Lake Conemaugh peaked hours before the dam breach. We therefore disagree with the main conclusion of Francis et al. (1891), that the dam failure was inevitable. They placed too much reliance on several estimates for the rate of rise of the water and the implied excess flow into the reservoir. This seems unusual given James Francis' expertise in flood

control and weir calculations and his practical knowledge of river behavior in flood, from the rapid rise to peak discharge and the relative speed of flood recessions. The committee assumed flow depths in the spillway that were implausible, based on evidence from the dam remnants that they themselves visited (i.e., preserved plow furrow on crest). Their unsupported assumption of protracted, extreme lake inflow before and after the dam breach led to their conclusion about the non-survivability of the dam.

We believe the investigation report represented state-of-the-art in 1891, with review of the embankment design and history, careful surveys and photographic documentation of the dam site, hydrologic analyses, and the inclusion of alternate views of the dam failure and site characteristics. However, we are puzzled that the 1891 conclusions are not consistent with evidence easily available to the committee. Their report (Francis et al., 1891) contains no reference to a second spillway. We find it difficult to believe that the original auxiliary spillway at the southwest abutment was unknown to the committee, given their review and documentation of the engineering specifications for the dam. They visited the dam site at a time when the original excavations on the western abutment should still have been apparent, and, most importantly, obtained a detailed post-flood survey of the dam remnants (Fig. 6 and Fig. 7) that clearly showed the southwest abutment was lower than the dam crest, even after the entire crest had been lowered by the SFFHC. In fact, Francis commented that " . . . near the ends [of the dam] there were ascents to the level of the top of the dam." (Francis et al., 1891, p. 468). The committee also received a report of the magnitude of flow that occurred over the left abutment after the digging of a shallow ditch, before flow had overtopped the crest. Yet, it appears none of the committee members perceived this clear evidence of an auxiliary spillway which would have functioned effectively given the original, higher dam breast. This spillway would have been more apparent before the construction of a parking lot at the western abutment circa 1977.

The confidence with which the committee stated the dam would have failed in any event was unwarranted. They made no comment about the smaller riprap used on the downstream face of the dam, even though the size difference was obvious in photographic Plate LIIIA of their report. The committee went so far as to conclude that had the dam been rebuilt to its original height, the dam breach could possibly have been more disastrous due to the larger impounded volume. They even suggested that lowering the dam widened the crest, and that its use as a road must have increased its resistance to breaching. But any improvement in resistance from using the crest as a road would have been insignificant compared to the unparalleled benefits of maintaining the higher crest. The combination of increased storage and more than doubled discharge capacity would have prevented overtopping long enough to protect the South Fork dam.

The ASCE review of the South Fork dam (Francis et al., 1891) and supporting calculations appear to us to have been biased in favor of the dam owners, thereby helping to shield the SFFHC, in a historic engineering sense, from subsequent liability claims or even the perception of liability in the wake of the disaster. We believe an injustice was thereby done to the more than 2200 people who lost their lives and to the survivors. The ASCE publication by Frank (1988) provided an updated perspective on the cause of the flood, but did not include supporting hydraulic calculations. J. Wesley Powell (1889), director of the U.S. Geological Survey, wrote "The Lesson of Conemaugh" in which he discussed the importance of dams and factors to consider in their design, including basin analysis and gaging of precipitation and streams. The disaster thereby brought new attention to the safety of existing and future dams, undoubtedly saving many lives in the future.

Declarations

Author contribution statement

Neil M. Coleman, Uldis Kaktins: Conceived and designed the experiments; Performed the experiments; Analyzed and interpreted the data; Contributed materials, analysis tools or data; Wrote the paper.

Stephanie Wojno: Performed the experiments; Analyzed and interpreted the data; Contributed materials, analysis tools or data.

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The authors declare no conflict of interest.

Additional information

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