Hydrologic Modeling of the May 1889 South Fork Dam Failure

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Abstract

On May 30th, 1889, a festive Memorial Day in Johnstown, Pennsylvania was cut short by showers in the late afternoon. The rain continued all night and brought local rivers to the top of their banks by sunrise the next day. The rains did not relent, and the rivers continued to rise reaching the highest ever recorded stage in the City of Johnstown on May 31st.

Fifteen miles upstream, past a few small towns located at stops along the Pennsylvania Railroad (PRR), was the South Fork Dam. As the rivers crested, the spillway capacity was greatly exceeded, and the dam overtopped. While Johnstown was hopeful the worst had passed, the dam was beginning to unravel. Despite human intervention to dig a new emergency spillway and raise the crest, the downstream slope eroded through the dam crest eventually resulting in a breach and emptying the lake in about 45 minutes. The flood wave covered the 15 miles to Johnstown in about an hour. It collapsed the PRR Viaduct bridge downstream, as well as many smaller bridges on its way to Johnstown. However, the recently completed stone PRR bridge at Johnstown resisted the flood wave creating a massive pile of debris that covered several acres. The bridge pile up included most of the town's wood framed buildings, rail cars, and much more. By evening the debris caught fire resulting in an estimated 80 additional fatalities. Overall, roughly 30,000 people were impacted resulting in at least 2,200 fatalities.

The authors of this paper discuss how they developed and calibrated a 2D HEC-RAS model of the dam breach and flooding prior to the breach. Data collection included: terrain, hydrology, high water marks, and flood arrival times gathered from historic photos, interviews and other published works. Development and calibration of the 2D HEC-RAS model will be discussed in detail. Historic observations are compared to the model results. A brief history of the flood and consequences is also covered.

Introduction

Background

On May 30th, 1889, a festive Memorial Day in Johnstown, Pennsylvania was cut short by showers in the late afternoon. The rain continued all night and was bringing local rivers to the top of their banks when the sun rose on May 31st. Johnstown lies at the confluence of the Little Conemaugh and Stony Creek Rivers where they combine to form the Conemaugh River. The little bit of flat terrain in the Allegheny Mountains was heavily developed by 1889 but had experienced repeated flooding every couple of years. On May 31st the rains did not relent, and the rivers continued to rise reaching the highest ever recorded stage in the City of Johnstown. Some areas were under 10 feet of water by lunch time. Most homes were 2 or more stories so people that didn't shuttle their families to shelter on the steep hillsides, moved the furniture and family upstairs. They sat there most of the afternoon unable to move without a boat, waiting for the water to recede. Around 3 PM it seemed to have peaked.

Fifteen miles upstream, past a few small towns located at stops along the Pennsylvania Railroad (PRR), was the South Fork Dam. The dam was completed in 1852 on South Fork Creek, a tributary to the Little Conemaugh River, and was the highest embankment dam in the country at the time. It was built to supply water to canals as part of the Portage Railroad's scheme to transport people and goods over the Allegheny Mountains. Constructed over 14 years in fits and starts, it was completed the same year it became obsolete when PRR finished the rail line that would put the Portage Railroad out of business. The dam failed without significant consequence after a minor flood in 1862 due to a "foundation defect" and lay in a breached state for 17 years until it was bought by a land developer. Benjamin Ruff wanted to rebuild the dam and turn the reservoir into an exclusive hunting and fishing club. Once established, the club paid to repair the dam. Unfortunately, it was not rebuilt with any engineering input and neglected many of the precautions taken by the original designers. As the rivers crested, the spillway capacity was greatly exceeded, and the dam overtopped. While Johnstown was hopeful the worst had passed, the dam was beginning to unravel. Despite human intervention to dig a new emergency spillway and raise the crest, the downstream slope eroded through the dam crest eventually resulting in a breach and emptying the lake in about 45 minutes. People downstream had heard tales of the dam breach for years and were mostly unconcerned by the telegraphed news of the worsening condition. The flood wave covered the 15 miles to Johnstown in about an hour. When it arrived, the pile of debris pushed by the flood raised a cloud of smoke and scraped all but the strongest masonry and concrete buildings down to their foundations. It collapsed the 80-foot tall PRR Viaduct. However, the recently completed stone PRR bridge at Johnstown resisted the flood wave creating a massive pile of debris that covered several acres. It included most of the town's wood framed buildings, rail cars, and much more. By evening the debris caught fire resulting in an estimated 80 additional fatalities. Overall, roughly 30,000 people were impacted resulting in at least 2,200 fatalities.

The US Army Corps of Engineers (USACE) Risk Management Center (RMC) is responsible for development of the consequence estimating procedures for the USACE Dam and Levee Safety Programs including the LifeSim software. The RMC is interested in validating the LifeSim model against real-world dam and levee failures and chose the failure of the South Fork Dam near Johnstown, PA as a test case. The RMC considered these specific circumstances worthy of investigation because it is the largest dam failure life loss event in US history. The timing and geographic extent of the case also made model development feasible. HDR was tasked by the RMC with collecting information and building a case history for LifeSim validation that includes an HEC-RAS hydraulic model of the May 1889 event to be used by USACE in a LifeSim model they develop. This paper only summarizes the HEC-RAS model development and hydraulic results.

Data Collection

Data were collected by HDR from many sources including photos, books, articles, and witness testimony to support the development and calibration of the hydraulic model. The most important sources are noted in the References and cited throughout. The most relevant calibration data were arranged geographically. There was a surprising amount of data and commentary available from a disaster more than 130 years ago, although some was sensational according to the style of reporting at the time. More reliable, detailed or factual accounts were given priority. Agreement between multiple sources of information was sought such as a photo and a testimony.

Data collection was focused on establishing hydrology (inflows to the dam as well as downstream tributaries), breach parameters, and hydraulic calibration points. Three types of calibration points were made: a maximum depth, depth at a specific time, and arrival time of the breach flood wave. Arrival time can be hard to determine for a slowly rising flood, but most people described the leading edge of the breach flood wave as a "wall of water" pushing debris in front of it and/or raising the water level very rapidly. This phenomenon would be the most recognizable during the flood to pinpoint at a specific time.

Hydrology and Hydraulics

HDR developed an HEC-RAS 2D unsteady hydraulic model of the South Fork Dam breach and subsequent Viaduct Bridge breach based on the best available topography data, collected historic data, and previous studies. Breach parameters were developed based on analysis of photos, drawings, and previous studies (Kaktins et al., 2013, Coleman et al., 2016). The model was used to simulate the South Fork Dam breach, the subsequent Viaduct breach, and the flood inundation along Conemaugh River from South Fork Dam to 5 miles downstream of Johnstown Stone Bridge.

HEC-RAS Model Development

Table 1 summarizes the terrain data, boundary conditions, model criteria, parameters, and assumptions. Figure 1 shows the model domain.

Table 1. Model Configuration Summary: HEC-RAS 2D Model

Parameter or feature	Description
Model and version	Fully-2D unsteady HEC-RAS model This analysis uses HEC-RAS 6.2, released March 2022.
Vertical datum	NAVD88: • NAVD88 = 1889 dam survey elevations + 6.2 ft. The conversion factor is used to convert information reported in 1889 measurements (Coleman et al, 2016) to NAVD88 (the model datum).
Topography and terrain data	The terrain data was acquired from Pennsylvania State (PASDA). It is a bare earth DEM with 2.5 ft resolution derived from the 2019 USGS PA Western LiDAR data set. It was originally in NAD_1983_2011_Pennsylvania_South_ftUS coordinate system but projected into USA_Contiguous_Albers_Equal_Area_Conic_Feet for modeling purposes. Terrain modification patches were developed to remove the present bridge approach embankments of US-219 near the South Fork Dam site.
Model domain	The model extends 5 miles from the Conemaugh Gap up the Conemaugh River to Johnstown and the confluence of the Little Conemaugh and Stonycreek Rivers. From there the Little Conemaugh River extends 14 miles up to Summerhill, PA. At South Fork, PA the South Fork of the Little Conemaugh River extends from the confluence 2 miles to the former South Fork Dam. Five miles of the Stony Creek is included from Johnstown up to Ferndale, PA.
2D Mesh development	One 2D flow area is configured and it consists of 51,540 cells. The nominal grid size is 50 ft in the channels and 100 ft in the floodplain. A sensitivity test of grid size in the floodplain was performed to evaluate 50 ft, 100ft, and 200 ft grid sizes. Varying the mesh grid size in the floodplain area does not significantly affect the computed results, indicating the base mesh grid size of 100 ft in the flood plain is appropriate.

Parameter or feature	Description
Boundary conditions (hydrology)	Inflow boundary conditions are configured at: • Lake Conemaugh • Little Conemaugh River • Stony Creek River Inflow Boundary conditions are also configured at two tributaries in the model domain: • Sandy Run • Clapboard Run The selection of inflow boundary conditions is part of the model calibration effort that considered the flood in Johnstown area before the dam failure and the time when the dam breached. The water levels were observed to increase from morning to noon on May 31, 1889. The inflow into Lake Conemaugh was configured based on the reported hydrograph in the hydrology study by Coleman et al, 2016. The inflows at other locations were set to be steady or increase from 6:00 am to 10:00 am and then be steady after 10:00 am the morning of the breach. Flow exits the model through the downstream normal depth outflow boundary located along Conemaugh River 5 miles downstream of Johnstown Stone Bridge. The outflow boundary friction slope is 0.00145 (estimated from water surface slope). A friction slope sensitivity test has been performed and shows that varying the outflow boundary friction slope does not affect the computed results in Johnstown or the upstream area, indicating the outflow boundary is sufficiently downstream from the reach of interest to eliminate influence. An initial lake level was set to 1604.1 ft, NAVD88. Based on the inflow hydrograph, this lake level set at 6:00 am results in the observed water surface elevation at 2:25 pm, just at the time the breach occurred. Six hours initial condition time was applied in the 2D area to simulate the base flow conditions in the Conemaugh River, Little Conemaugh River, Stony Creek River, and other tributaries at 6:00 am on May 31, 1889. Water was mostly contained in the river channels and there was minor flooding in Johnstown area at that time.

Parameter or feature	Description
Hydraulic Structures	Three major hydraulic structures are configured as 2D connection structures in the model: • South Fork Dam: The former South Fork Dam was configured as two 2D connection structures: 1) the spillway east of the dam embankment and 2) the main dam embankment section that failed in 1889. The dam was modelled as two separate structures in order to have different weir coefficients for water going over the rock-based channel/spillway (C=1.5) versus over the dam crest (C=2.6). The main spillway was not really a designed concrete spillway structure, but rather a rock excavated channel. • Viaduct Bridge: The Viaduct Bridge was configured as a 2D connection structure with a 65 ft tall “gate” representing the actual arch opening. This structure was not modeled as a 2D bridge because it would be breached, and breaching is currently not available for a 2D bridge in HEC-RAS. To account for the observed debris from the South Fork Dam breach flood wave that blocked the arch opening, the “gate” is configured half open (32.5 ft) and gradually closed when the bridge starts to fail (this was done to not double account the flow going through the breach, as the main opening was part of the breach failure area). HEC-RAS does not account for a gate being part of the breach opening. • Johnstown Stone Bridge: Johnstown Stone Bridge was configured as a 2D bridge. The deck and pier geometry parameters are selected to approximately represent the 7 arch openings. The opening widths are adjusted to account for the bridge skew from flow direction. Severe debris was observed upstream of Stone Bridge and hence 60 ft wide and 10 ft high floating debris is configured to each pier. All other bridges in the model domain were destroyed early in the event. Therefore, no other bridges are modeled.
Hydraulic equation set	Full momentum (SWE-ELM equation set)
Computation time step	Fixed time step of 1 sec
Event simulation time window	18 hours, from May 31, 1889, 06:00 to May 31, 1889, 24:00.
Model assumptions and limitations	The topography and land cover information during the dam failure in 1889 is not available and we are aware that some major changes were made since the dam failure, including the channel improvement and realignment in the Johnstown area. However, the flood dramatically overwhelmed the channel capacity. Peak depths\water surface elevations and arrival times were compared to available data and found to be reasonable for the purpose of consequence modeling.

<table>
<tr><th>Parameter or feature</th><th>Description</th></tr>
<tr><td>Manning's n values</td><td>Manning's n values were assigned based on the earliest available NLCD 2001 land cover database and the literature (Chow, 1959 & USACE, 2016):
<table>
<tr><th>Land Cover</th><th>Roughness</th></tr>
<tr><td>barren land rock/sand/clay</td><td>0.025</td></tr>
<tr><td>cultivated crops</td><td>0.035</td></tr>
<tr><td>deciduous forest</td><td>0.15</td></tr>
<tr><td>developed, low intensity</td><td>0.06</td></tr>
<tr><td>developed, medium intensity</td><td>0.08</td></tr>
<tr><td>developed, high intensity</td><td>0.12</td></tr>
<tr><td>developed, open space</td><td>0.04</td></tr>
<tr><td>emergent herbaceous wetlands</td><td>0.05</td></tr>
<tr><td>evergreen forest</td><td>0.1</td></tr>
<tr><td>grassland/herbaceous</td><td>0.035</td></tr>
<tr><td>mixed forest</td><td>0.15</td></tr>
<tr><td>open water</td><td>0.03</td></tr>
<tr><td>pasture/hay</td><td>0.04</td></tr>
<tr><td>shrub/scrub</td><td>0.07</td></tr>
<tr><td>woody wetlands</td><td>0.07</td></tr>
</table>
NOTE: Manning's n values in the main river channels were refined and assigned to 0.03, 0.035, and 0.04 as determined from aerial imagery and channel bed slopes.</td></tr>
<tr><td>Storage area</td><td>Lake Conemaugh (also known as the South Fork Reservoir and Western Reservoir) was configured as a 1D storage area connected to the downstream 2D flow area by South Fork Dam.

The lake stage-storage curve was input from a study by Coleman et al. (2016). It was developed using LiDAR data and historic observations about the lake, dam, and breach.</td></tr>
<tr><td>Rainfall and infiltration</td><td>Not included.</td></tr>
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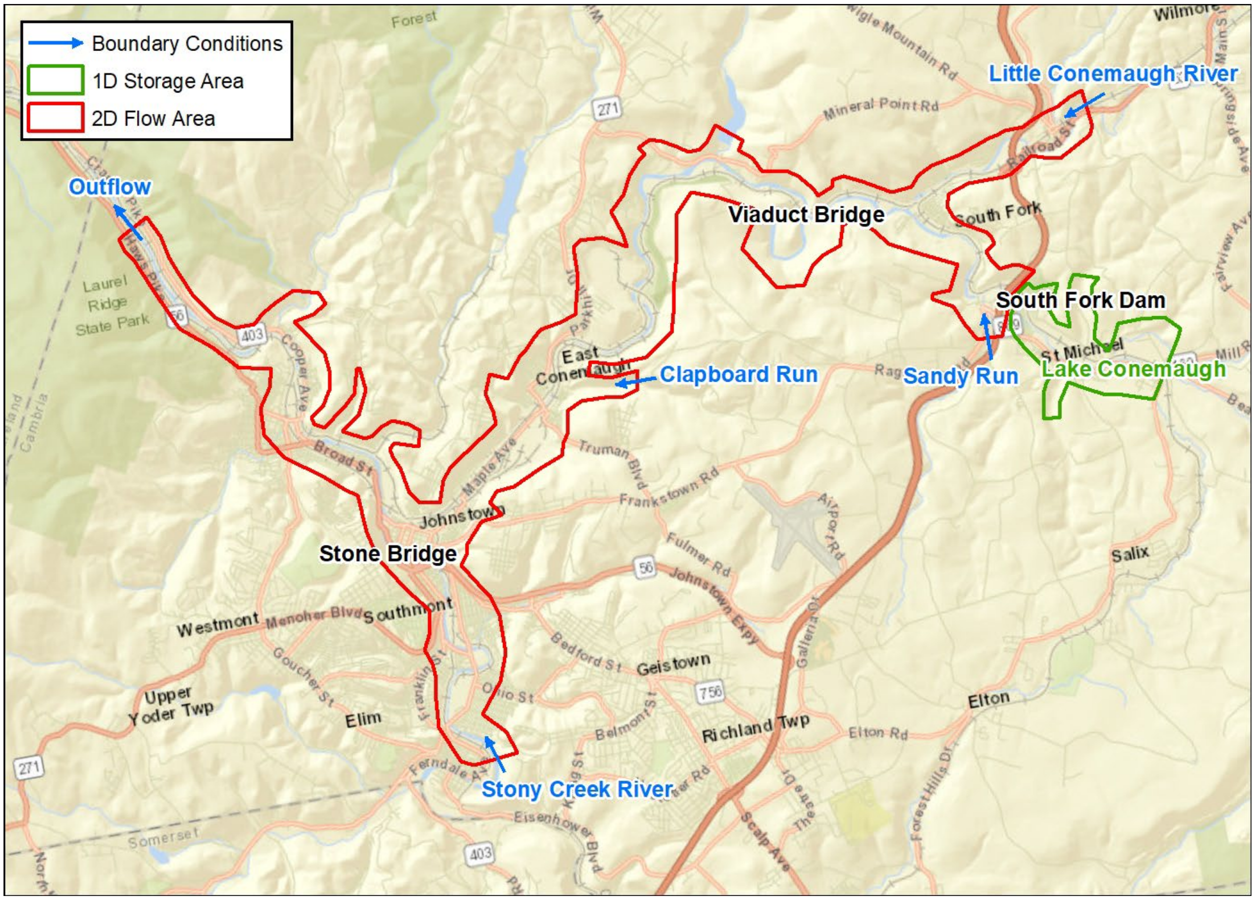

Figure 1. HEC-RAS 2D model extent and locations of boundary conditions.

The South Fork Dam and Viaduct Bridge were configured to breach, and the breach parameters were developed based on collected information (photos, surveys, studies, etc.). The failures were set to start based on an observed water surface elevation for South Fork Dam and an estimated water surface elevation for the Viaduct Bridge. Table 2 and Table 3 list the breach parameters. Figure 2 and Figure 3 show the diagrams of breach geometry. Figure 2 shows a photo of the dam after the breach. Additionally, there is a diagram showing the dimensions of the breach (Coleman, 2016). Below the picture and diagram is the HEC-RAS modelled trapezoidal breach. HEC-RAS only allows a single trapezoidal cut for the breach. The breach dimensions for the South Fork Dam were based on the plot that was developed from surveying the breach area after the flood event. As the surveyed breach flow area was a complex shape, a trapezoidal breach was fit to the opening in order to obtain the same breach flow area at the water surface trigger elevation (Table 2). The breach dimensions for the Viaduct Bridge (Figure 3 and Table 3) were estimated based on reviewing photos taken after the flood (Coleman 2016).

Table 2. Breach parameters for South Fork Dam

Parameter or feature	Description
Failure mode	Overtopping
Trigger criteria	Water surface elevation: 1,616.3 ft, NAVD88
Breach bottom elevation	1,546.9 ft, NAVD88
Breach bottom width	67.25 ft
Side slope of breach	2.5 H:V
Breach weir coefficient	2.6
Breach formation time	0.25 hr

Table 3. Breach parameter for Viaduct Bridge

Parameter or feature	Description
Failure mode	Overtopping
Trigger criteria	Water surface elevation: 1,460 ft, NAVD88
Breach bottom elevation	1,384 ft, NAVD88
Breach bottom width	110 ft
Side slope of breach	0.8 H:V
Breach weir coefficient	2
Breach formation time	0.2 hr

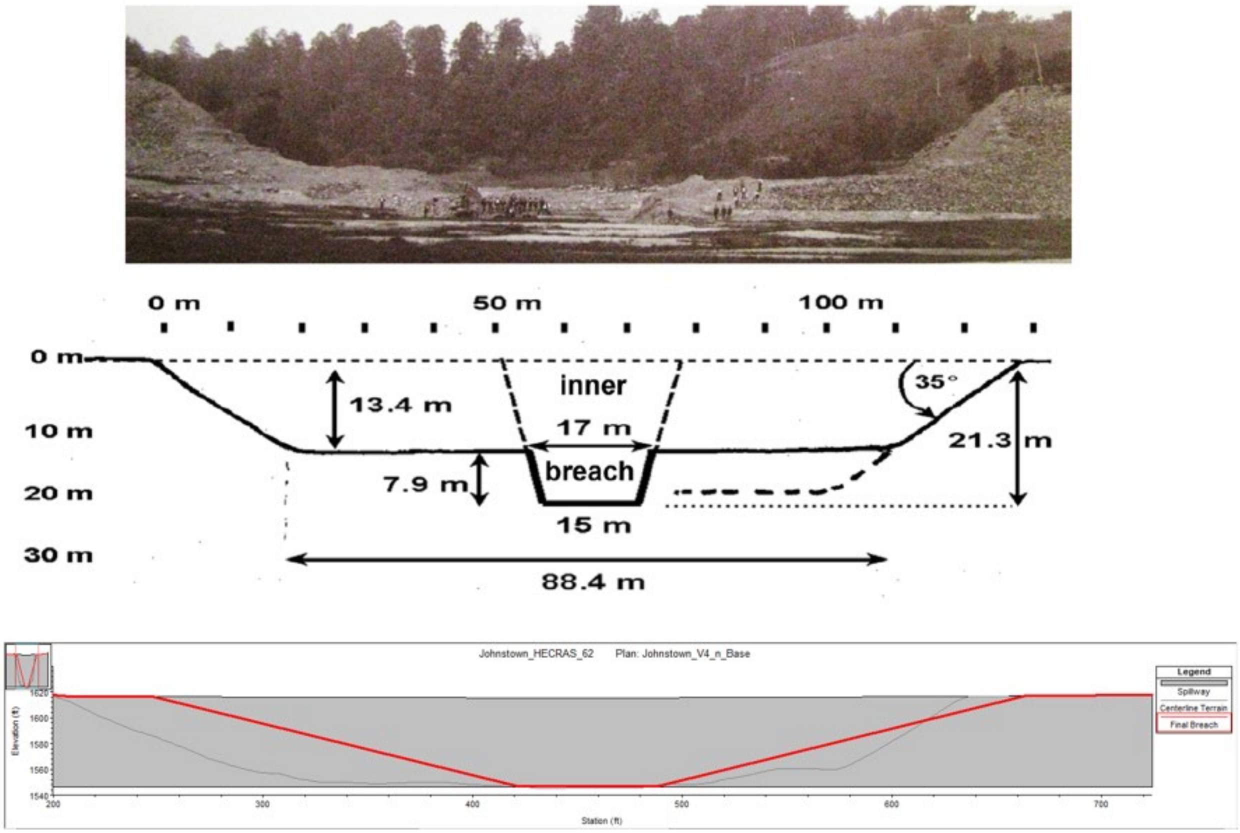

Figure 2. Ultimate breach geometry and photo for South Fork Dam (Coleman, 2016).

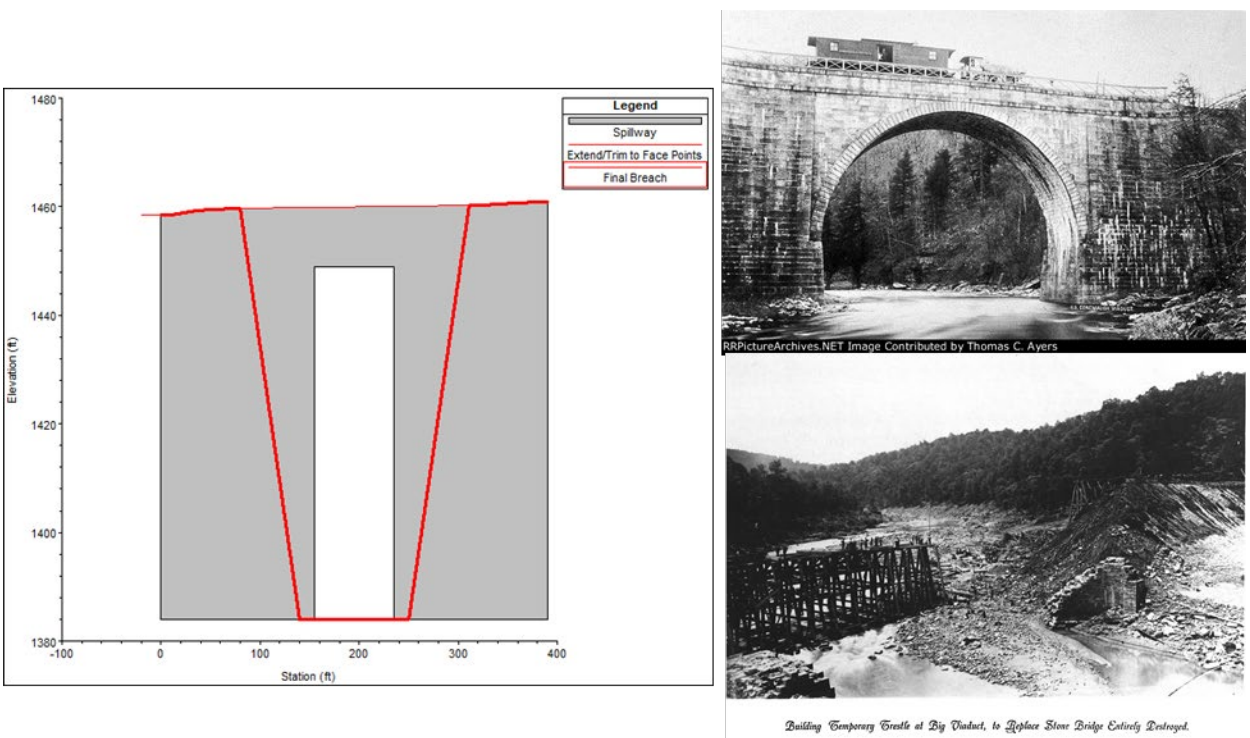

Figure 3. Ultimate breach geometry for Viaduct Bridge with before and after photos.

HEC-RAS Model Calibration

Various data were collected and reviewed to calibrate the HEC-RAS hydraulic model. These data start from early morning on May 31, 1889, before the failure of the South Fork Dam and go until after the breach flood wave passed the Johnstown area in late afternoon. It should be noted that most of the collected information is anecdotal statements and thus the accuracy and reliability cannot be guaranteed. Because much of the information was anecdotal, the calibration process did not try to hit every high-water mark closely, but rather fit the data that was considered most reliable first.

The data sources include:

- Anecdotal statements/surveys by local residents, railroad employees, or officials
- Photos by journalists after the flood and others prior to the event
- Previous Johnstown Flood studies

The major calibration locations are:

- South Fork Reservoir (Lake Conemaugh)
- South Fork Dam
- Viaduct Bridge
- Johnstown Stone Bridge
- Water depth/Max water depth at various times/locations along Little Conemaugh River and in the Johnstown area
- Flood wave arrival time at various locations along Little Conemaugh River and in the Johnstown area

The primary calibration parameters are inflow hydrographs at Lake Conemaugh (South Fork Reservoir), the Little Conemaugh River at Summerhill, PA and the Stony Creek River at Ferndale, PA, and breach parameters of South Fork Dam and Viaduct Bridge. Manning's roughness was also adjusted along the channel to refine the flood wave arrival time. A lower roughness will allow for faster flow, but at a lower depth, requiring a balancing of accuracy between arrival time and depth. Given the lack of gage data or reliable flow data at the inflow locations, various flows were tested and determined based on the collected observation data. Figure 5 shows the calibration locations used in this analysis. The calibration data and modeling results are detailed in Table 4. Observed depths are converted to water surface elevations based on the model terrain. Figure 4 shows a profile of the modeled maximum water surface elevations along Little Conemaugh River versus the observations. Additionally in Figure 4 is a graph showing the computed water surface minus the observation values in the form of a water surface difference. Figure 5 shows the model calibration points in a map view for reference.

Table 4. Model calibration results and observations

ID	Description	Type	Observation	Results
1	South Fork Dam	Breach time	Breach occurred between 14:45 to 15:00	Breach occurs at 14:45
2	South Fork Cr Wagon Bridge	Depth	EL 1,476.3 ft at 10:00	1,475.6 ft at 10:00
3	South Fork PRR Depot	Arrival time/Max depth	EL > 1,502.4 ft (max) Flood wave arrival at 15:08	1,504.4 ft (max) Flood wave arrival at 15:08
4	SO Tower - South Fork	Arrival time/Max depth	*EL > 1,506.4 ft (max) Flood wave arrival at 15:05	1,499.5 ft (max) Flood wave arrival at 15:10
5	Viaduct Gap	Max depth	EL 1,475.7 ft (max)	1,474 ft (max)
6	Big Viaduct	Max depth	EL 1,463.7 ft (max)	1,463.5 ft (max)
7	AO Tower	Arrival time/Max depth	EL > 1,312.8 ft (max) Flood wave arrival at 15:40	1,308.7 ft (max) Flood wave arrival at 15:45
8	Conemaugh Roundhouse	Max depth	EL 1,242.3 ft (max)	1,241 ft (max)
9	Conemaugh Tower	Arrival time	Flood wave arrival at 15:48	Flood wave arrival at 15:54
10	Woodvale Flour Mill	Max depth	EL 1,205.3 ft (max)	1,206.6 ft (max)
11	James Quinn House	Arrival time/Depth	EL 1,174.5 ft shortly before 16:00 Flood wave arrival shortly before 16:00	1,174.2 ft shortly before 16:00 Flood wave arrival at 16:15
12	City Hall - 1907	Max depth	EL 1,185.1 ft (max)	1,186 ft (max)
13	Rev Beale's Parsonage*	Depth	EL 1,180.5 ft at 16:00	1,173.9 ft
14	Cambria Club House	Max depth	EL 1,183.2 ft (max)	1,185.9 ft (max)
15	TH Watt house	Arrival time/Depth	EL 1,176 ft at 15:30 Flood wave arrival at 16:07	1,173.4 ft at 15:30 Flood wave arrival at 16:12
16	Dibert St & Morris St	Depth	EL 1,174.6 ft at 15:30	1,174.3 ft at 15:30
17	St. Mary's Church	Max depth	EL 1,170.1 ft (max)	1,170.2 ft (max)
18	Wagner House*	Max depth	EL 1,172.7 ft (max)	1,169.8 ft (max)

1. Times are on May 31, 1889 and elevations are in NAVD88.
2. * denotes points not used in calibration due to poor data quality.

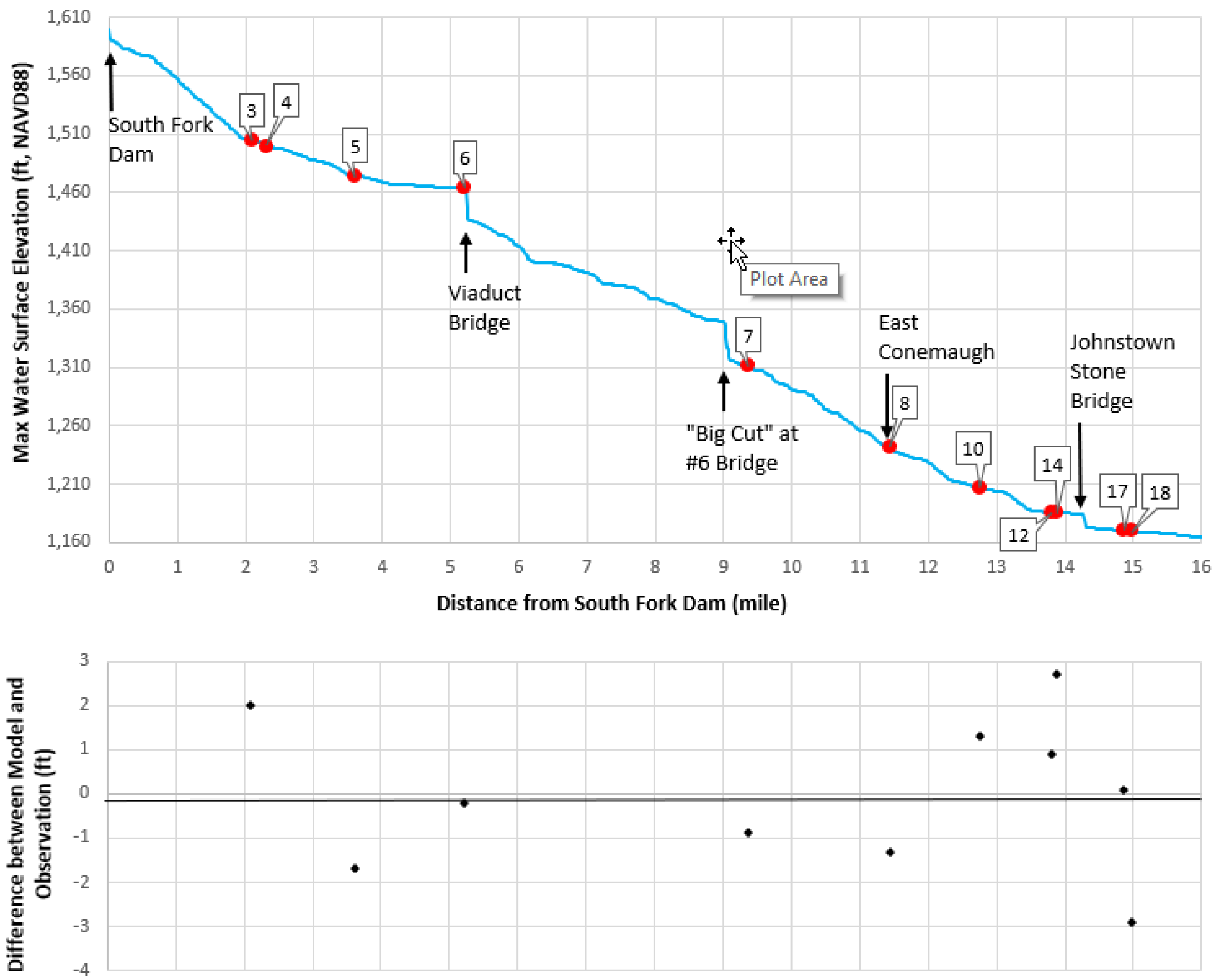

Figure 4. Maximum water surface elevation calibration results and observations

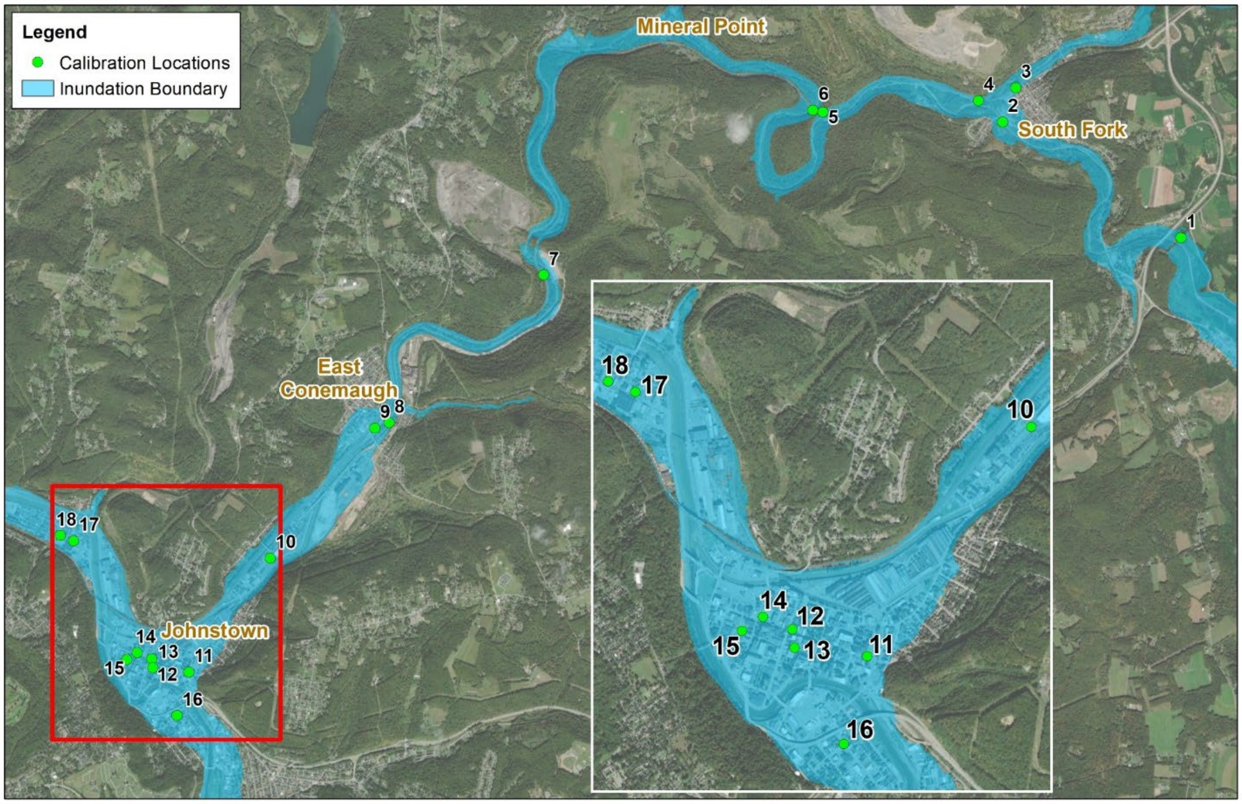

Figure 5. Model calibration locations

Conclusions

Analysis of the calibration results shows that the model was able to reproduce the travel time of the event quite well (See times in Table 4), as travel times were less than 15 minutes from reported values. Computed minus reported water surfaces varied across the modeling domain. In some locations the model results were high, in others they were low (as shown in Figure 4). As mentioned previously, as most of the observations are from anecdotal reports, the model was calibrated to the observations where the data was deemed to be more accurate (i.e. photo evidence, etc…). The resulting water surface elevations shown in Table 4 and Figure 4 are deemed to be very reasonable given the inconsistencies in the reported values. For example, the water surface reported at point 18 is 2.6 feet higher than the water surface reported at point 17, even though point 17 is relatively close and is also upstream of point 18.

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