Note:

The following pages contain two attachments referred to the request for correction at https://www2.usgs.gov/info_qual/documents/Initial_inquiry_021016.pdf.

For complete documentation refer to https://www2.usgs.gov/info_qual/1972_PA_streamflow_data.html.

Attachment 1 (pages 3 to 29)

HE2769

Reanalysis of Flood of Record Using HEC-2, HEC-RAS, and USGS Gage Data

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Abstract: Three independent analyses establish that the Conestoga River flow published by the U.S. Geological Survey for Tropical Storm Agnes in 1972 of 1 420 cubic meters per second (50 300 cubic feet per second) should have been at least 1 660 cubic meters per second (58 600 cubic feet per second), an increase of over 16 percent. The three analyses included an empirical analysis of the data for U.S. Geological Survey gage 01576500, a re-analysis of a 1978 HEC-2 simulation of the Conestoga River, and a retrofit and minor corrections to a preliminary 2013 HEC-RAS simulation of the Conestoga River.

Introduction

The 1972 tropical storm (U.S. Geological Survey and the National Oceanic and Atmospheric Administration, 1975) resulting from Hurricane Agnes (Agnes) caused the flood of record at the U.S. Geological Survey gage 01576500 at the city of Lancaster, Pennsylvania (Figure 1). The U.S. Geological Survey determined in 1990 that the peak flow was 1 420 cubic meters per second, or 50 300 cubic feet per second. Because the gage failed during the flood, the U.S. Geological Survey estimated the peak flow using observed high water marks and hydraulic analysis described by Benson and Dalrymple (1967). The hydraulic analysis included the control for gage 01576500 located 18 meters (60 feet) downstream--the five-arch stone Viaduct shown in Figure 2. The hydraulic capacity of the rightmost arch (looking downstream) was altered beginning in October 1990 when the Pennsylvania Department of Transportation replaced an existing unpaved road located a meter or so above the normal water level by the four-lane Pennsylvania Highway 23 (East Walnut Street) built approximately 3 meters higher than the unpaved road and occupying the entire base of the arch.

The U.S. Geological Survey stream gage 01576500 <http://waterdata.usgs.gov/pa/nwis/uv/?site_no=01576500&PARAmeter_cd=00065,00060,00010> is located on the left bank looking downstream. In Figure 2 the gage housing is obscured by a few trees. The gage datum is at an elevation of 74.868 meters (245.63 feet). The upstream drainage area is 839 square kilometers (324 square miles). The gage datum is defined with National Geodetic Vertical Datum (NGVD) of 1929, with the conversion from NGVD 1929 to the North American Vertical Datum (NAVD) 1988 of -0.25 meters (-0.82 feet) (Federal Emergency Management Agency, 2005). A weir across the third and fourth arches from the east bank may have been constructed to pool water upstream of the Viaduct, because the Lancaster City water intake is located approximately 100 meters upstream. At an elevation of 75.13 meters (246.50 feet) NGVD 1929, the weir is level and 1.2 meters above the low point in the river-bed, according to a 1978 HEC-2 simulation (Roy F. Weston, Inc., 1978).

The Geological Survey determination of the peak flow from Agnes was complex, but illustrates the uncertainty in estimating these large peak flows by indirect methods (Benson and Dalrymple, 1967). Because Agnes flooded and shut down gage 01576500, the U.S. Geological Survey initially estimated the peak flow as 2 500 cubic meters per second (88 300 cubic feet per second) (U.S. Geological Survey and the National Oceanic and Atmospheric Administration, 1975) based on a water surface height of 8.47 meters (27.8 feet) (unpublished revision request dated August 4, 1989) after the 1972 flood, and this was the flow published in Federal Emergency Management Agency (1980). A HEC-2 simulation (Boswell Yule Jordan Engineering, 1989) determined a new flow for Agnes of 1 150 cubic meters per second (40 500 cubic feet per second) based on HEC-2 (U.S. Army Corps of Engineers-HEC, 1990, revised 1991) water surface profiles computed in January of 1989. In response to the August 4, 1989 revision request, Flippo (unpublished Revision Comments, 1990) first revised the 1972 peak flow to 1 690 cubic meters per second (59 600 cubic feet per second). After further consideration, the Geological Survey revised the 1972 peak flow to 1 420 cubic meters per second (50 300 cubic feet per second) and revised the 1972 peak water surface elevation to 8.59 meters (27.9 feet) pending construction of the four lane highway through the westernmost arch of the Viaduct.. Flippo (unpublished Revision Comments, 1990) did not mention the Boswell Yule Jordan Engineering estimate of 1 150 cubic meters per second (40 500 cubic feet per second) in his Revision Comments. The peak flow and peak water surface elevation have not been revised since 1990.

This paper reports the independent re-analysis of this vital flood of record and recommends a further increase in flow based on three independent sources of information. These sources are;

(1) The July 17, 2015 U.S. Geological Survey rating curve for gage 01576500.

(2) A HEC-2 hydraulic analysis done during 1978 for the U.S. Federal Emergency Management Agency.

(3) A preliminary 2013 HEC-RAS hydraulic analysis (unpublished) also for the U.S. Federal Emergency Management Agency.

Analysis of Gage 01576500 Rating Curve

The U.S. Geological Survey updates the Lancaster and all other gage stage-discharge curves periodically. Figure 3 presents the July 17, 2015 relationship from http://waterdata.usgs.gov/nwisweb/data/ratings/exsa/USGS.01576500.exsa.rdb and also shows 85 annual peak flows from http://nwis.waterdata.usgs.gov/pa/nwis/peak/?site_no=01576500&agency_cd=USGS beginning with 1929 and ending with the annual peak flow for 2014. Although the gage record spans 86 years, the annual peak flow for 1932 did not include a water surface height and could not be plotted. The rating curve in Figure 3 is weighted to lower water surface elevations that caused the relationship to significantly deviate from a power relationship Rantz S.E. et al. (1982). The 1929, 1930, and 1931 peak flows are outliers not included in further analyses.

The June 23, 1972 flood of record is plotted in Figure 3 based on the final 1990 estimate, having a water surface height of 8.59 meters (27.9 feet) and flow of 1 420 cubic meters per second (50 300 cubic feet per second). The second highest maximum annual water surface height occurred September 8, 2011 when water 6.64 meters (21.3 feet) above the gage datum also flooded and shut down the gage; the indirect estimate of flow was 855 cubic meters per second (30 200 cubic feet per second). Figure 3 shows that the pre-1990 annual peak flows (the 57 diamond points not including the annual peak flows from 1929 through 1932) mostly plot above the present curve. The average amount by which the pre-1990 measured flows exceed those obtained from the the July 17, 2015 rating curve is 9 percent. The average amount by which the 1990 through 2014 annual peak flows (the 24 square points) plot below the curve is 2.7 percent. The algebraic difference of 11.7 percent makes sense because the four lane highway built in 1990 reduced the open area of the stone Viaduct, and a given flood height since 1990 corresponds to a lower flow, as shown by the newer square points.

Using engineering judgement the author determined a lower limit for the flows shown in Figure 4 by plotting only those annual peak flows equaling or exceeding a 2.33-year flood of 250 cubic meters per second (8 700 cubic feet per second). This study determined the magnitude of a 2.33-year flood using the Expected Moments Algorithm (Cohn, et al, 1997) in the PeakFQ computer program (Flynn, et al., 2006). The input for PeakFQ included the final estimate for the 1972 annual peak flow (Agnes) of 1 420 cubic meters per second (50 300 cubic feet per second) and included all annual peak flows from 1933 to 2014.

This 2.33-year flow is significant for two reasons:

1. It is the mean annual flood for gage 01576500 (Dalrymple, 1960).
2. The corresponding water surface height from the July 17, 2015 rating curve is 3.48 meters (11.42 feet), which approximately equals the elevation of the published flood stage for the gage, 3.4 meters (11 feet).

The 2-year flow using the same EMA analysis was 220 cubic meters per second (7 800 cubic feet per second). Had this been the lower cutoff it would have added three additional annual peak flows (1951, 1967, and 1975) to the power curve, however for all three flows the measured water surface height was from 0.18 meters (0.6 feet) to 0.24 meters (0.8 feet) below flood stage.

The discrete rating curve <http://waterdata.usgs.gov/nwisweb/data/ratings/exsa/USGS.01576500.exsa.rdb> passes directly through the 2011 and the 1972 flows. Because of the 1990 decrease in cross section, the 1972 flow should exceed the July 17, 2015 rating curve along with the other maximum annual flows observed prior to 1990. Alternatively the Lancaster rating should show corrections for each stage-discharge measurement prior to 1990. To extrapolate the 1972 flood of record from annual maximum flows prior to 1990, a power curve $Q = 58.927\ S^{2.0751}$ (long dashed line in Figure 4) was the best fit of that portion of the annual maximum series.

This study also fit the short dashed power curve $Q = 58.496\ S^{2.00401}$ to evaluate the indirect 2011 flow estimate from the post-1990 portion of the annual maximum series. The solid line of discrete points in Figure 4 is the July 17, 2015 U.S. Geological Survey rating curve <http://waterdata.usgs.gov/nwisweb/data/ratings/exsa/USGS.01576500.exsa.rdb> for flows exceeding 2 50 cubic meters per second (8 840 cubic feet per second). This study did not use the round data points for the 1972 and 2011 floods of record to fit the power curves, thus avoiding bias in evaluating the extrapolations of the series of the pre- and post 1990 annual maximum flows.

This study extrapolated the short dashed power curve equation fit to eleven square data points representing annual maximum floods from 1994, 1996, 1997, 1999, 2000, 2003, 2004, 2006, 2007, 2013, and 2014 to estimate a flow for the 2011 flood of 850 cubic meters per second (30 000 cubic feet per second) at a water surface height of 6.64 meters (21.3 feet). This extrapolated flow is very close to the U.S. Geological Survey estimate of 855 cubic meters per second (30 200 cubic feet per second). The correlation coefficient (R^2) is 0.99.

Results – 1972 Flood

This study extrapolated the long dashed power curve equation fit to twenty square data points representing the annual maximum floods, exceeding the 2.33-year flood, beginning with 1933 and ending with 1989. The estimated flow for the 1972 flood was 1 670 cubic meters per second (58 900 cubic feet per second) at a water surface height of 8.50 meters (27.9 feet). The correlation coefficient (R^2) is 0.95. This extrapolated flow compares favorably with the initial U.S. Geological Survey revision to the Agnes flow, estimated by Flippo (unpublished Revision Comments, 1990) to be 1 690 cubic meters per second (59 600 cubic feet per second).

The analysis of the U.S. Geological Survey gage rating curve was an empirical first approximation indicating that a peak flow for Agnes could have exceeded the 1990 estimate of 1 420 cubic meters per second (50 300 cubic feet per second). Two other ways to check the flow for Agnes were available. Roy F. Weston, Inc. (1978) published the HEC-2 analysis. Dewberry, Inc. (unpublished, 2013) prepared a preliminary HEC-RAS analysis. This study repeated the 1978 backwater simulation and 2013 simulation using HEC-RAS (U.S. Army Corps of Engineers-HEC, 2010a, 2010b) to ensure comparability. The reanalysis of the 1978 backwater simulation and check of the 2013 preliminary simulation required the hydraulic characteristics of the Conestoga River Viaduct before 1990, and a boundary condition downstream of the Viaduct.

1978 Back Water Analysis

The 1978 HEC-2 simulation was based on the National Geodetic Vertical Datum (NGVD) of 1929, the same datum for the complete record still being measured at U.S. Geological Survey gage 01576500. The conversion to the North American Vertical Datum (NAVD) of 1988 is NGVD 1929 – 0.25 meter (- 0.82 feet) = NAVD 1988. The river stations began at the confluence of the Conestoga River with the Susquehanna River, approximately 35 kilometers downstream of gage 01576500. The 1978 and 2013 river stations differ by about 122 meters (400 feet) in the vicinity of the gage. A scan of the original HEC-2 simulation input file is in Auxiliary File 1 (Weaver, 2016A). The source of this file was the Federal Emergency Management Agency Engineering Library in Alexandria Virginia. Auxiliary File 2 (Weaver, 2016B) is the same HEC-2 input file in ASCII text format ready for import into HEC-RAS. Auxiliary File 3 (Weaver, 2016C) includes the HEC-RAS files produced after the HEC-2 input file was imported into HEC-RAS.

High-water Mark

Even though the U.S. Geological Survey gage failed during Agnes, the high-water mark inside and outside the gage housing was established by the U.S. Geological Survey in 1990 to be 8.50 meters (27.90 feet), which, when added to the gage datum of 74.87 meters (245.63 feet), produced a water surface of 83.37 meters (273.53 feet) NGVD 1929.

The Conestoga River Viaduct

The arches of the Viaduct are 16.5 meters (54 feet) wide as entered in the 1978 HEC-2 simulation. The flow depth in Agnes averaged 4.3 meters (14 feet) in Arch 1 (East bank) and 6.6 meters (21.5 feet) in Arch 2. The slope distance of the ground surface between the arches is no more than 0.3 meter (1 foot) greater than the arch width, or no more than 2 percent greater. Considering two sides to each arch, the sides of the arches make up 37 percent of the wetted perimeter for Arch 1 and 43 percent for Arch 2. For Arches 3 and 4, the sides of the arches located above the weir accounted for 50% of the wetted perimeter during Agnes (60 percent if the starting point was the river bed). In Arch 5 the sides accounted for 43 percent of the wetted perimeter prior to construction of East Walnut Street.

Downstream Boundary Condition

Determining a starting water surface to use downstream of the flow gage was complicated by two ambiguities – the location and the correct elevation of the downstream boundary condition. Flippo (unpublished Revision Comments, 1990) wrote that the flow of record was "... computed on the basis of 71.60 feet for the tailwater". With no information in the record indicating where this "tailwater" was observed, this study used the most downstream cross section in the HEC-2 input file, at River Station 117+986 feet upstream of the mouth of the Conestoga River (see Auxiliary File 1 (Weaver, 2016A)). This cross section was located 975 meters (3 200 feet) downstream of gage 01576500. Identifying this as the most downstream cross section might seem counterintuitive unless viewed in context. In 1978, the detailed flood simulations often began and ended at municipal boundaries, or where sufficiently valuable property at risk of flood damage justified the additional expense of the detailed simulation. For the Conestoga River reach that included gage 01576500, the HEC-2 input file began 975 meters (3,200 feet) downstream of the flow gage, making that the most likely location for measurement of the tailwater immediately after the flood.

The second ambiguity regarding the tailwater elevation was more easily resolved. Because the U.S. Geological Survey gage datum is 74.87 meters (245.63 feet), the so-called tailwater elevation of 71.60 feet referenced by Flippo only made sense if he actually meant 271.60 feet, or 82.78 meters.

This effort then imported the 1978 HEC-2 file into HEC-RAS while retaining the NGVD 1929 elevations but changing the downstream boundary condition to 82.78 meters (271.60 feet). The author set the flow at 1 420 cubic meters per second (50 300 cubic feet per second) as estimated by Flippo (unpublished Revision Comments, 1990). The HEC-RAS simulated water surface elevation at the gage was 83.39 meters (273.58 feet) NGVD 1929, which was very close to the recorded high-water mark of 83.37 meters (273.53 feet). This also falls well within the expected tolerance of 0.15 meters (0.5 feet) when HEC-2 files are imported into and run in HEC-RAS (Federal Emergency Management Agency, 2002).

Simulating the observed high-water elevation at gage 01576500 provided excellent verification of the downstream boundary condition and indicated how the U.S. Geological Survey may have estimated the 1972 Agnes peak flow (Flippo unpublished Revision Comments, 1990). Despite this, some round 2 corrections appeared necessary for the 1978 HEC-2 input data.

The final revision to the initial simulation was to decrease the Manning coefficient of the main channel from 0.04 to 0.036 at cross section 184.010, located 139 meters (455 feet) downstream of the Viaduct. The HEC-RAS simulated water surface elevation at the gage was then 83.37 meters (273.53 feet) NGVD 1929, matching the recorded high-water mark at the gage and calibrating the simulation.

Additional Revisions to the 1978 HEC-2 Viaduct Modeling

In the 1978 simulation, Roy F. Weston, Inc. described the Viaduct using 5 separate cross sections, as shown in Figure 5. The cross section numbers used in the HEC-2 input file had no relation to the river station, so this study renumbered these sections in HEC-RAS; cross section 183.0, located 975 meters (3 200 feet) downstream of the Viaduct was redesignated cross section 1. The renumbered cross sections are listed in Table 1, along with the HEC-2 designation and a brief description of each. The weir spanning the third and fourth Viaduct arches became part of cross section 6.

Two issues with the 1978 HEC-2 simulation related to the internal Viaduct cross sections were the selections of dimensionless (1) expansion and contraction coefficients and (2) Manning roughness coefficients. In comparing the expansion and contraction coefficients from cross section 8 immediately upstream of the Viaduct to those for cross section 7 inside the Viaduct, the use of 0.6 and 0.8 for abrupt transitions (the local standard of care at the time (U.S. Army Corps of Engineers, 1990)) should likely have been 0.3 and 0.5, respectively, as is current practice (U.S. Army Corps of Engineers, 2010a).

This investigation also examined the 1978 expansion and contraction coefficients inside the Viaduct. Excepting the 1.2 meter (4 foot) high weir in arches 3 and 4 of cross section 6, these Viaduct cross sections have the same shape, yet in the 1978 HEC-2 model, the expansion and contraction coefficients were set to 0.3 and 0.5, respectively, between sections 5, 6, and 7. More appropriate expansion and contraction coefficients for cross sections 5 and 7 were 0.0 and 0.0 which indicate no or negligible transition losses. Because of the weir, this study selected minimal coefficients of 0.1 and 0.3 for cross section 6, despite the weir being submerged by over 7 meters of water during Agnes.

Immediately downstream of the Viaduct, the expansion and contraction coefficients were originally entered as 0.6 and 0.8 at section 4, again indicating an abrupt transition using 1978 practice. Using current practice (2015) these coefficients should have been 0.3 and 0.5, respectively, given the relative uniformity of the river cross sections. Table 1 lists revised expansion and contraction coefficients for the Viaduct.

The second correction to the Viaduct involved the Manning coefficients used to model surface roughness in the Viaduct cross sections. The 1978 specified cross sections 5 and 7 inside the Viaduct are shown in Figure 6 (cross section 6 is shown in Figure 8). As shown in Figure 6 within the Viaduct, the 1978 simulation (Roy F. Weston, Inc. 1978) varied the Manning coefficient between arches 3 and 4 bridging the main channel as 0.04 versus 0.08 for arches 1 and 2 on the east floodplain and 0.12 for part of the west floodplain under arch 5. Under arch 5 the Manning coefficient assigned to the dirt road was 0.01, 0.04 assigned to the floodplain to the east of the road, and 0.12 assigned to the slope west of the road. Other than the Manning coefficient of 0.04, none of the other values make sense within these stone arches.

Using the values in USDOT (1961) as guidance, along with Engineering judgement, this study used a Manning coefficient of 0.02 for the sides of the arches, which make up 30 to 60 percent of the wetted perimeter of each arch. This study used a Manning coefficient of 0.05 for the heavy weeds and scattered brush between Arches 1 and 2. This is at the low end of the suggested range of 0.05 to 0.07 (USDOT, 1961) but is reasonable considering the approximately

5 to 7 meter (16 to 23 foot) depth of flow between these arches during Agnes, which would have flattened any vegetation present. No trees or large shrubs were visible in Figure 7, which is an aerial photograph taken on July 5, 1971. As a result of these observations, the author changed the Manning coefficients to 0.04 for the entire width of each cross section within the Viaduct. These Manning coefficients were also in general agreement with those used by Dewberry (unpublished, 2013) for the internal bridge sections in their simulation.

After revising both the expansion and contraction coefficients and the Manning coefficients, the author re-ran the 1978 simulation in HEC-RAS with the downsteam water surface elevation of 82.78 meters (271.60 feet) presumed to have been observed at cross section 1, and using various flows. At a flow of 1 660 cubic meters per second (58 600 cubic feet per second), the computed flood stage matched the observed high-water elevation of 83.37 meters (273.53 feet) NGVD 1929.

Revisions of Preliminary 2013 HEC-RAS Hydraulic Analysis

Dewberry (unpublished, 2013) made the preliminary 2013 HEC-RAS simulation using the North American Vertical Datum (NAVD) of 1988, so instead of converting the entire HEC-RAS input file to NGVD 1929, the author instead chose to convert the final simulation results to NGVD 1929. In this preliminary HEC-RAS hydraulic simulation, Dewberry (unpublished, 2013) designated the Conestoga River Viaduct as a bridge rather than five cross sections. No cross sections were provided at the upstream and downstream faces of the Viaduct, as is standard practice, and no ineffective flow or floodplain dead zones of storage were specified.

Figure 8 compares the NGVD 1929 coordinates at the centerline of the Viaduct among the 1978 and 1989 HEC-2 simulations and the 2013 HEC-RAS simulation. The weir specifications in both the 1989 HEC-2 simulation and the 2013 HEC-RAS simulation show the weir (level in Figure 5) at higher elevations and with varying amounts of skew compared with the 1978 HEC-2 simulation. The weir specifications in the 1989 and 2013 simulations are thus erroneous. In the 100-year flood (Federal Emergency Management Agency, 1980) the effect of the erroneous weir specifications on the 1989 HEC-2 flow is only about a one and one half percent reduction compared with the weir specified in the 1978 simulation. Variations in the remainder of the cross section, other than East Walnut Street, may have been due to erosion and deposition in the intervening years. East Walnut Street is depicted by the black line in Figure 8, including the concrete barrier along the river side of the highway.

This study did not attempt to estimate the 1972 peak flow during Agnes using the 1989 HEC-2 simulation. To estimate the peak flow during Agnes using the 2013 HEC-RAS simulation, the author made the following revisions (all elevations adjusted to NAVD 1988):

1. Replaced the central portion of the cross section at River Station 120814, being the location of the U.S. Geological Survey flow gage 01576500, with the original configuration from cross section 185.000 of the 1978 HEC-2 simulation
2. Copied the revised cross section 120814 at the flow gage to become new cross sections at the upstream (River Station 120750) and downstream (River Station 120720) faces of the Viaduct as specified in the HEC-RAS General Modeling Guidelines (U.S. Army Corps of Engineers-HEC, 2010a), , set the expansion and contraction coefficients to 0.3 and 0.5 respectively (U.S. Army Corps of Engineers-HEC, 2010a and U.S. Army Corps of Engineers-HEC, 2010b), and added ineffective flow (U.S. Army Corps of Engineers-HEC, 2010a and U.S. Army Corps of Engineers-HEC, 2010b). The addition of ineffective flow only increased the computed water surface by 0.01 meters (0.03 feet), so the omission in the 2013 simulation had minimal effect.

3. Replaced the central portion of the 2013 HEC-RAS cross section at River Station 120667, 16 meters (53 feet) downstream of the Viaduct with the revised cross section from River Station 120814 because there was no equivalent cross section in the 1978 HEC-2 simulation. The central portions of the four cross sections from River Station 120814 to 120667 were identical.
4. Corrected both internal bridge sections to accurately simulate the interior of the Viaduct, including the weir and river bed, based on the 1978 HEC-2 cross sections. This included increasing the Manning coefficient under Arches 1 and 2 from 0.035 to 0.04, but leaving the Manning coefficients of 0.02 to 0.03 for the remaining arches as Dewberry entered them.
5. Although there was no effect on the water surface elevation at gage 01576500, the Author corrected an error in one of the HEC-RAS "Bridge Modeling Approach", "Low Flow Method" used by Dewberry. "Low Flow" is defined as flow that does not reach the lower chord of a bridge (U.S. Army Corps of Engineers-HEC, 2010a). Dewberry set their simulation to use the highest energy answer among the "Energy (Standard Step)" and the "Momentum" low flow methods. Dewberry initially set the pier drag for the "Momentum" method using a coefficient of 2.0 for square nosed piers. Figure 5 shows one triangular pier nose, with field observation revealing a mix of triangular and round noses. Based on suggested values in U.S. Army Corps of Engineers-HEC, 2010a, the author chose a coefficient of 1.39 for triangular nose with 60 degree angle. After correcting the pier drag coefficient and running the simulation, there was no change in the water surface elevation at the gage. This indicated that the "Energy (Standard Step)" method had produced the highest energy answer, and the results from the "Momentum"

method were irrelevant.

6. Although the preliminary 2013 HEC-RAS simulation extended to the Susquehanna River, this study truncated the simulation 975 meters (3 200 feet) downstream of the Viaduct at cross section 117559 to match the downstream limits of the 1978 HEC-2 simulation.
7. This study set a downstream boundary condition at cross section 117559 using a known water surface elevation of 82.53 meters (270.78 feet) NAVD of 1988 which equates to the 82.78 meter (271.60 foot) tailwater elevation based on the NGVD of 1929.

After making these revisions, this study interpolated the flow from HEC-RAS at the observed high-water elevation for Agnes adjusted to NAVD 1988 (an elevation of 83.12 meters or 272.71 feet). A flow of 1 660 cubic meters per second (58 700 cubic feet per second) matched the high water mark.

Conclusions

The estimated flow for Tropical Storm Agnes, determined in 1990, was 1 420 cubic meters per second (50 300 cubic feet per second) at the U.S. Geological Survey flow gage 01576500.

A power curve extrapolation of the largest 20 pre-1990 annual peak flows for water years beginning with 1933 and ending with 1989 (not inclusive) indicated the 1972 peak flow for Agnes was 1 670 cubic meters per second (58 900 cubic feet per second).

Using HEC-RAS, this study replicated the results of the 1978 HEC-2 simulation at gage 01576500. After correcting errors in the 1978 simulation, the estimated flow at the gage was 1 660 cubic meters per second (58 600 cubic feet per second). The Viaduct was simulated in 1978 using five cross sections.

This study also included the preliminary 2013 HEC-RAS simulation currently under preparation for Federal Emergency Management Agency (as of 2015). The preliminary HEC-RAS simulation specified the Viaduct as a standard HEC-RAS bridge using the post-1990 cross section. In order to use this simulation to estimate the 1972 peak flow for Agnes, the author replaced the 2013 Viaduct cross sections with the pre-1990 stations and elevations, added bounding cross sections at the upstream and downstream faces of the Viaduct, added ineffective flow, and corrected the HEC-RAS bridge parameters and internal cross sections. After these revisions, the estimated flow at the gage was 1 660 cubic meters per second (58 700 cubic feet per second).

Averaging the flows determined by the 1978 HEC-2 simulation run in HEC-RAS and the 2013 HEC-RAS simulation produced a flow of 1 660 cubic meters per second (58 600 cubic feet per second) for the flood of record, an increase of over 16 percent above the U. S. Geological Survey estimated flow.

Pending peer review of this paper, the author intends to pursue the U.S. Geological Survey Quality Assurance process for gage 01576500.

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References

Benson, M.A. and Dalrymple, T. (1967). *General Feld and Office Procedures for Indirect Discharge Measurements*. U.S. Geological Survey Techniques of Water-Resources

Investigations, Book 3, Chap. A1, Washington, D.C.

Boswell Yule Jordan Engineering (1989). S.R. 6023 (L.R. 1124), Section B01, *Hydrologic and Hydraulic Report for Relocating PA Route 23 over Conestoga River and an Unnamed Tributary to Conestoga River, Lancaster County, Prepared for Pennsylvania Department of Transportation*. Camp Hill, PA.

Cohn, T. A., Lane, W. L., and Baier, W. G. (1997). "An algorithm for computing moments-based quantile estimates when historical flood information is available." *Water Resour. Res.*, 33(9), 2089-2096.

Dalrymple, T. (1960). *Flood-Frequency Analyses, Manual of Hydrology: Part 3. Flood Flow Techniques.* Geological Survey Water-Supply Paper 1543-A, Washington, D.C. p. 4.

Federal Emergency Management Agency (1980). *Flood Insurance Study, Township of East Lampeter, Lancaster County, Pennsylvania.* Washington, D.C., p. 6.

Federal Emergency Management Agency (2002). "Floodplain Modeling Manual, HEC-RAS Procedures for HEC-2 Modelers." Appendix A. Washington, D.C., Appendix A.

Federal Emergency Management Agency (2013). *Flood Insurance Study, Lancaster County, Pennsylvania, All Jurisdictions.* Volume 1 of 4, Washington, D.C. p. 48, preliminary.

Federal Emergency Management Agency (2015). "Lower Susquehanna Watershed Map." <http://www.riskmap3.com/sites/default/files/lower_susquehanna_watershed_map.pdf> (December 13, 2015).

Flynn, K. M., Kirby, W., and Hummel, P. R. (2006). “User’s manual for program PeakFQ, annual flood frequency analysis using Bulletin 17B guidelines.” U.S. Geological Survey Tech. Methods, 4-B4, Reston, VA.

Rantz S.E. et al. (1982). *Measurement and computation of streamflow: Volume 2. Computation* *of Discharge.* U.S. Geological Survey Water Supply Paper 2175, Washington, D.C. pp. 330-331.

Roy F. Weston, Inc. (1978). "Final Floodway Upper Conestoga Creek". Federal Emergency Management Agency Engineering Library, Alexandria, VA. HEC-2 input file.

U.S. Army Corps of Engineers, Hydrologic Engineering Center (1991). *HEC-2 Water Surface Profiles, User's Manual.* Davis, CA.

U.S. Army Corps of Engineers, Hydrologic Engineering Center (2010a). *HEC-RAS River Analysis System, Hydraulic Reference Manual.* Version 4.1, Davis, CA.

U.S. Army Corps of Engineers, Hydrologic Engineering Center (2010b). *HEC-RAS River Analysis System, User's Manual.* Version 4.1, Davis, CA.

U.S. Department of Transportation (1980). *Design Charts for Open Channel Flow.* Hydraulic Design Series No. 3, Washington D.C.

U.S. Geological Survey and National Oceanic and Atmospheric Administration (1975). *Hurricane Agnes Rainfall and Floods, June-July 1972*. Geological Survey Professional Paper 924, Washington, D.C. p. 6.

Weaver, A. (2016A). Auxiliary File 1 for Reanalysis of Flood of Record Using HEC-2, FEC-RAS, and USGS Gage Data. figshare. <https://dx.doi.org/10.6084/m9.figshare.2060448.v1>. Retrieved: 17 29, Jan 04, 2016 (GMT).

Weaver, A. (2016B). Auxiliary File 2 for Reanalysis of Flood of Record Using HEC-2, FEC-RAS, and USGS Gage Data. figshare. <https://dx.doi.org/10.6084/m9.figshare.2060439.v1>. Retrieved: 17 18, Jan 04, 2016 (GMT).

Weaver, A. (2016C). Auxiliary File 3 for Reanalysis of Flood of Record Using HEC-2, FEC-RAS, and USGS Gage Data. figshare. <https://dx.doi.org/10.6084/m9.figshare.2060451.v1>. Retrieved: 17 40, Jan 04, 2016 (GMT).

Figure Captions List

Figure 1. Hatched area shows the Conestoga River basin upstream of U.S. Geological Survey stream gage 01576500. Base map from the Federal Emergency Management Agency (2015). HUC is Hydrologic Unit Code. (modified from FEMA)

Figure 2. Conestoga River Viaduct for rail traffic looking downstream, 1999. Library of Congress, Prints & Photographs Division, HAER, Reproduction number PA,36-LANC,10—1.

Figure 3. Stage-discharge relationships for U.S. Geological Survey stream gage 01576500, Lancaster, Pennsylvania. Gage benchmark is 74.87 meters (245.63 feet) National Geodetic Vertical Datum of 1929. Annual peak flows for 1929, 1930, and 1931 are outliers and are shown in gray. The annual peak flow record for 1932 did not include a water surface elevation and was omitted. Gage data courtesy of the U.S. Geological Survey.

Figure 4. Stage (S)-discharge (Q) relationships for U.S. Geological Survey stream gage 01576500 Lancaster, Pennsylvania. Gage benchmark is 74.87 meters (245.63 feet) National Geodetic Vertical Datum of 1929. Gage data courtesy of the U.S. Geological Survey.

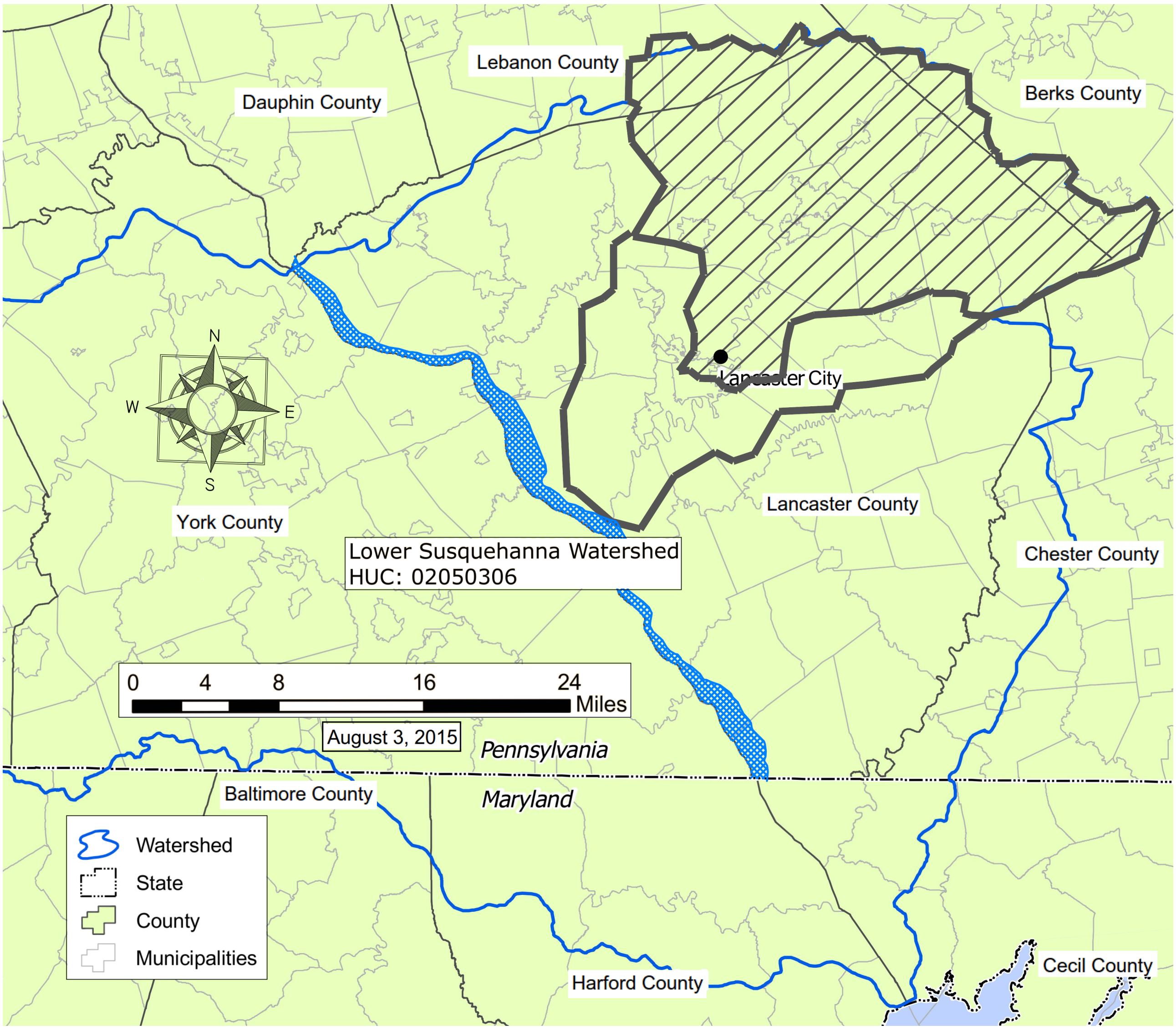
Figure 5. Arch 4 of the Conestoga River Viaduct. 1972 High Water Elevation = 83.372 meters (273.53 feet) NGVD 1929. 2011 High Water Elevation = 81.360 meters (266.93) NGVD 1929. Elevations shown were recorded at gage 01576500 located 18 meters (60 feet) upstream. Each block is approximately 0.5 meters (1.5 feet) high. (image by author)

Figure 6. Approximation of the Conestoga River cross sections 5 and 7 from Roy F. Weston, Inc. (1978).

Figure 7. July 5, 1971 aerial photo by the U.S. Department of Agriculture Farm Service Agency,

sponsored by the Pennsylvania Geological Survey on the PennPilot website (www.pennpilot.psu.edu). North is towards the top of the photo. Direction of Conestoga River flow is shown by the arrow.

Figure 8. Approximations of Conestoga River cross section 6. Weir (solid gray line) from Roy F. Weston, Inc. (1978) is located at elevation 75.13 meters (246.5 feet) NGVD of 1929.



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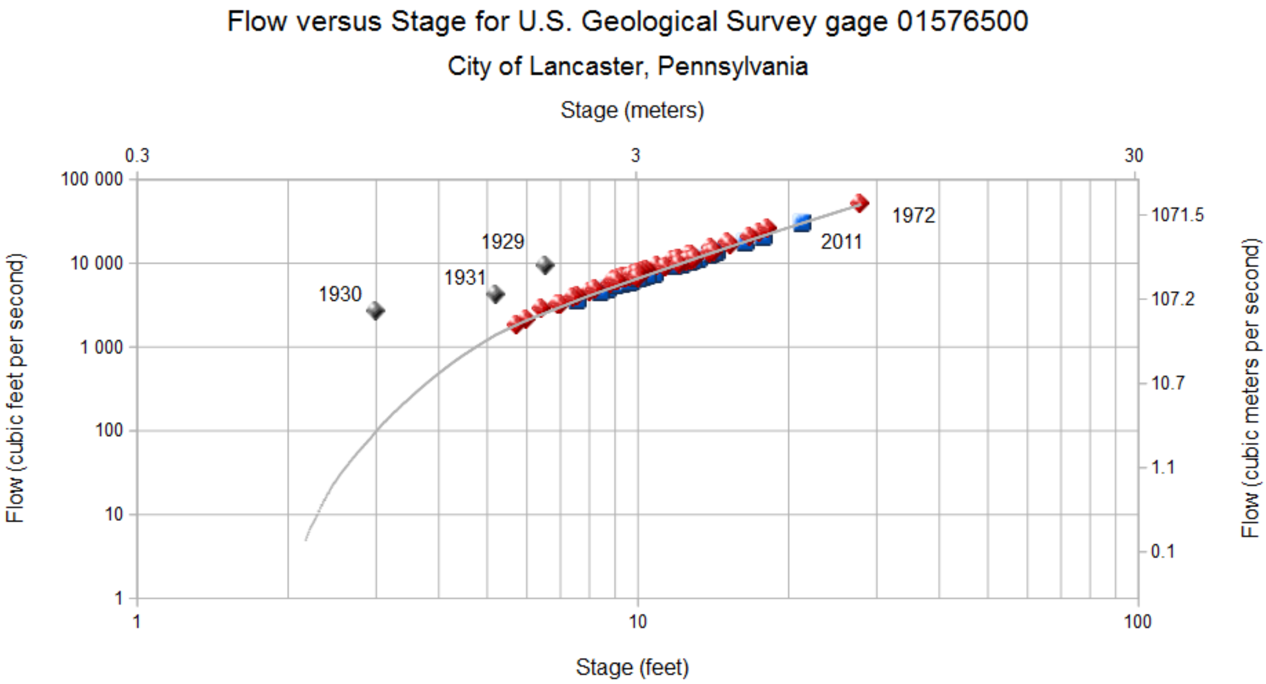

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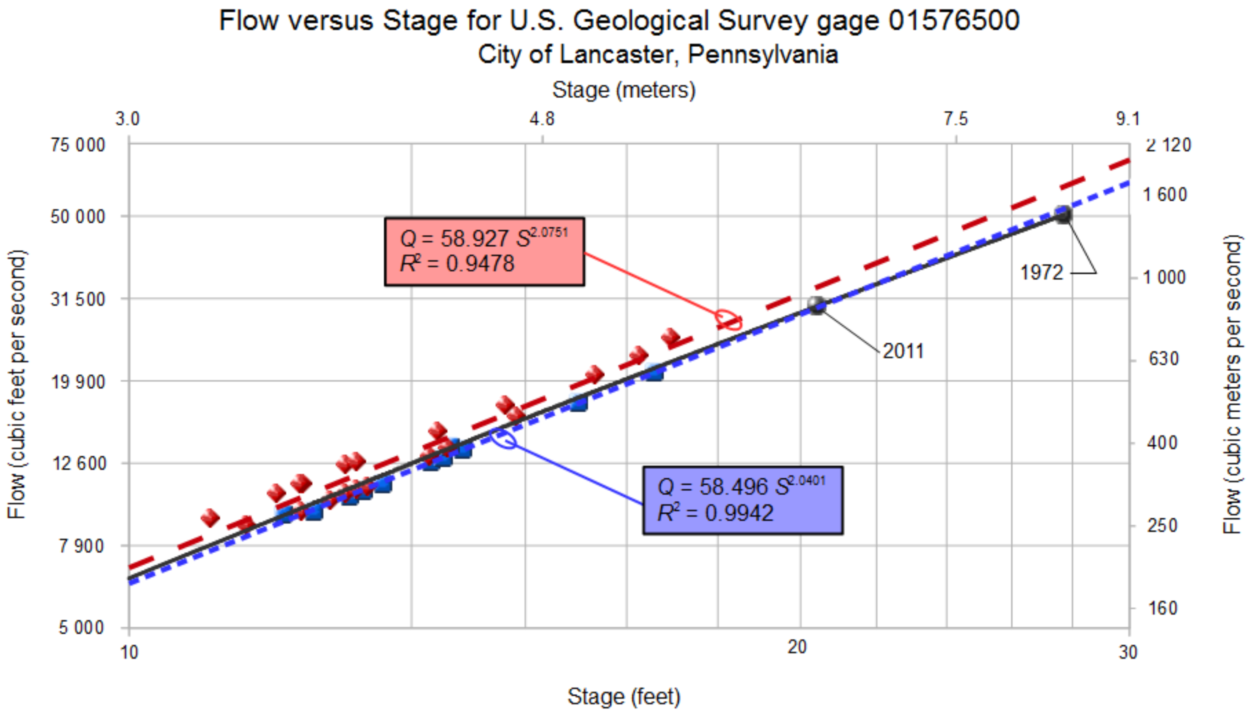

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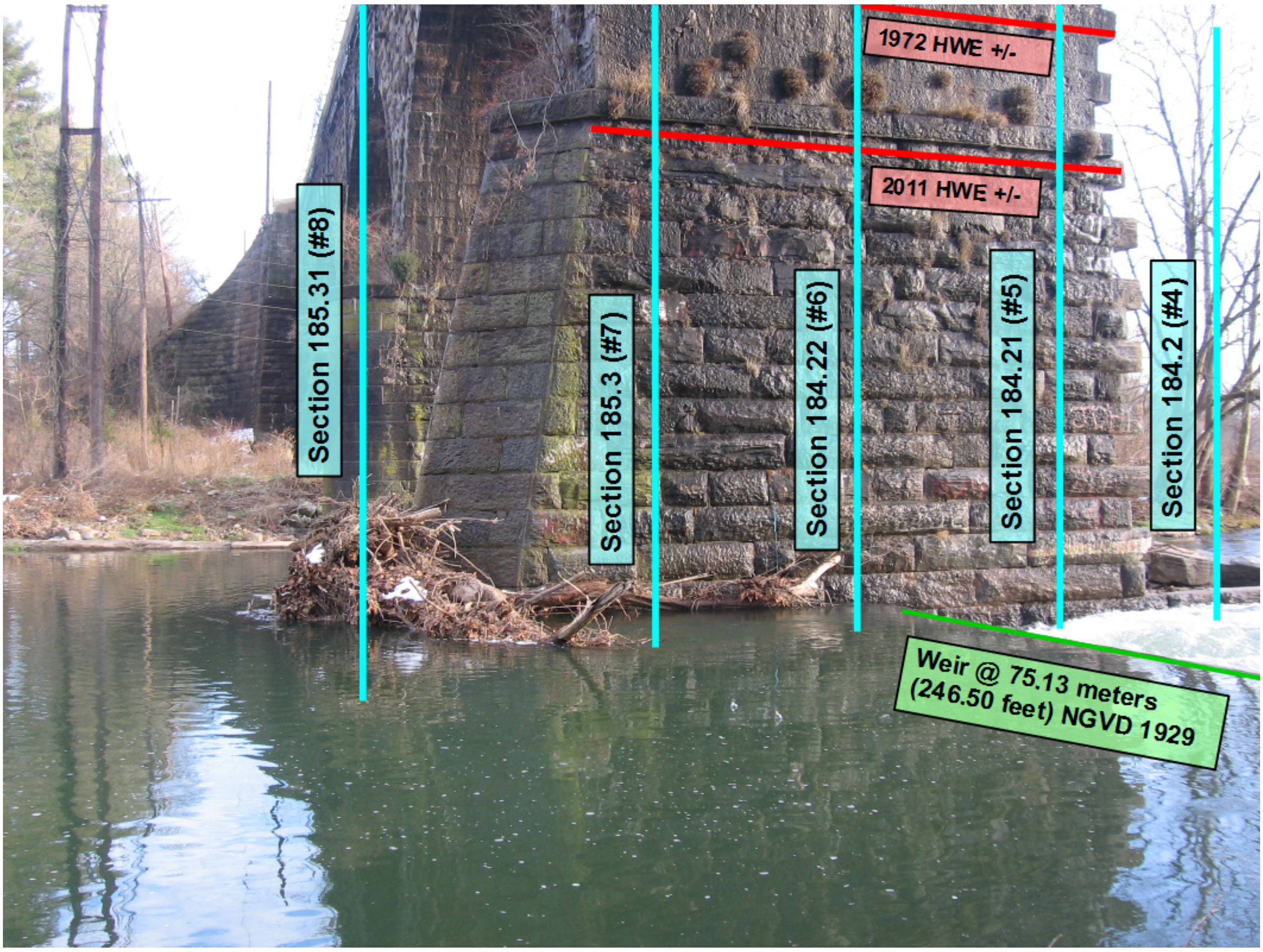

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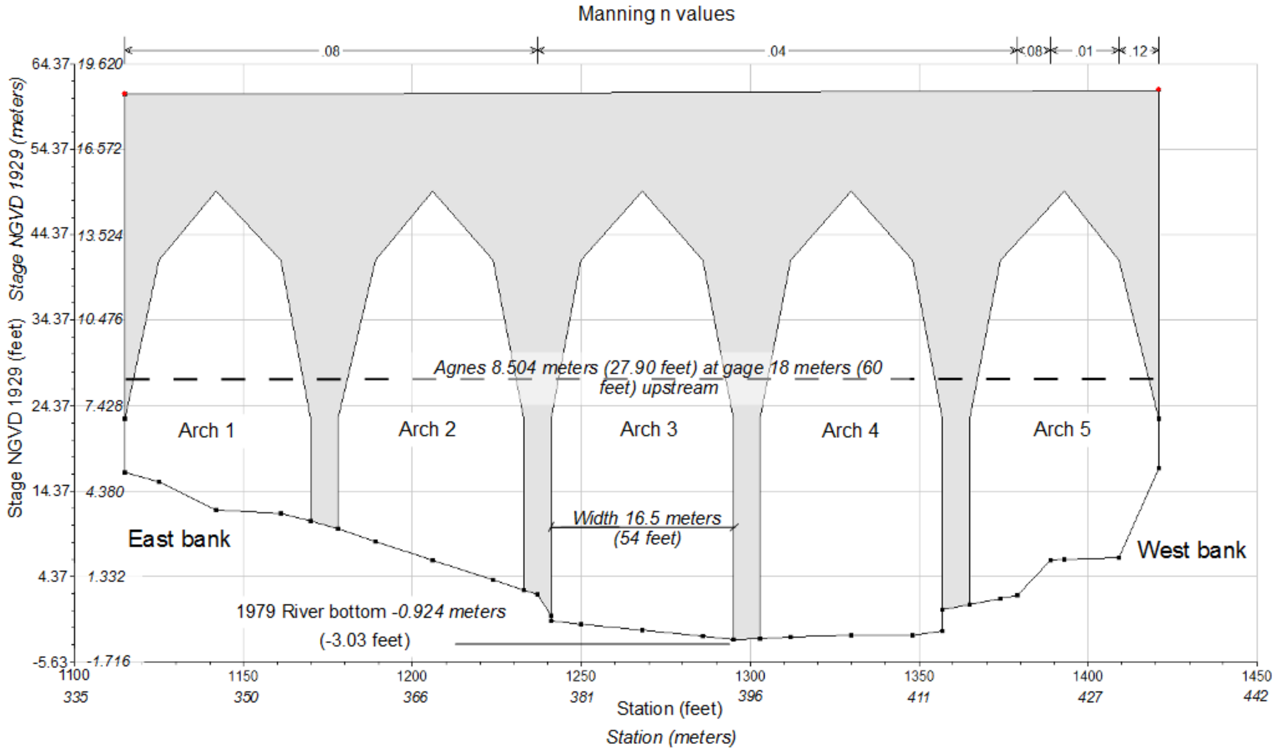

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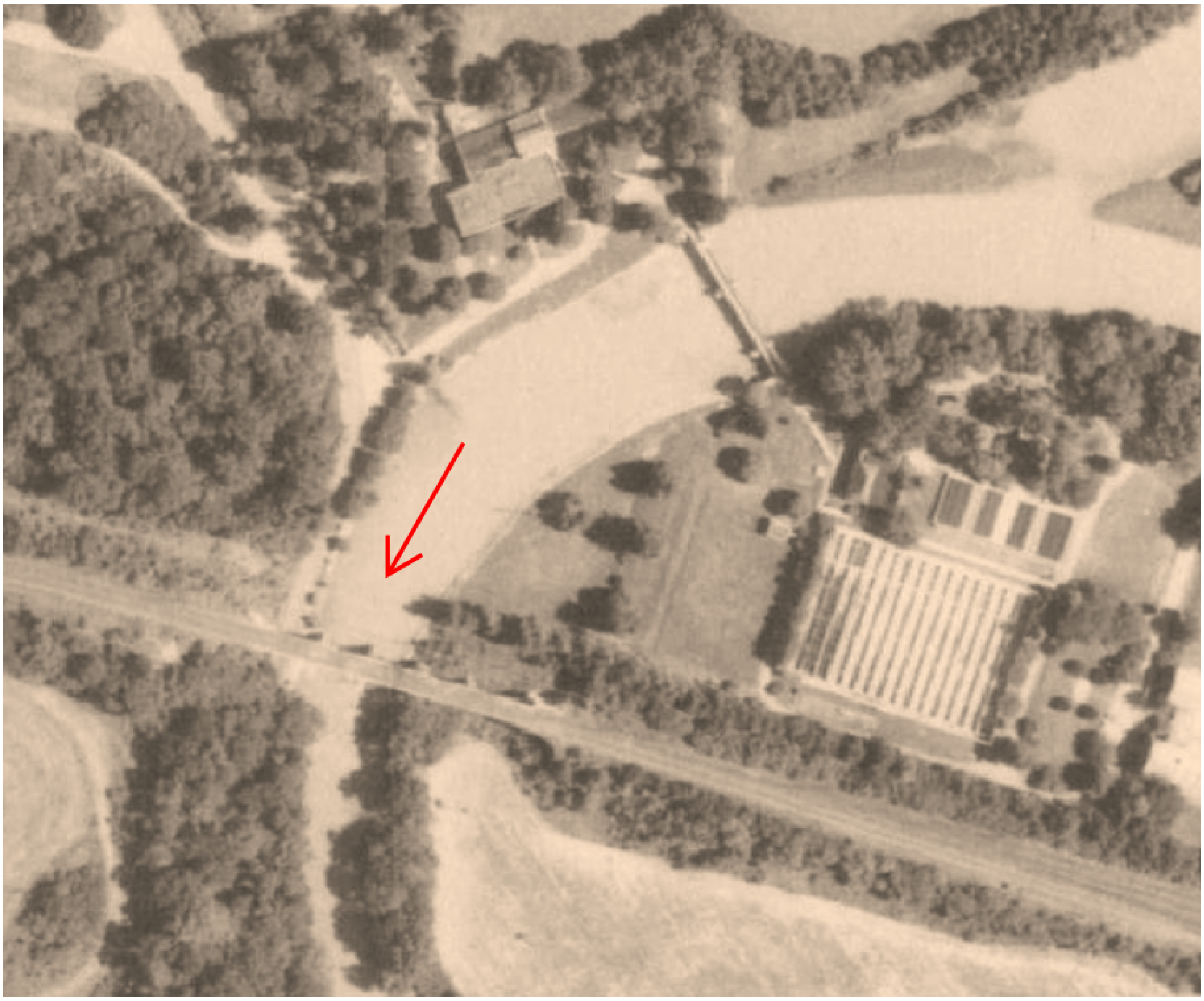

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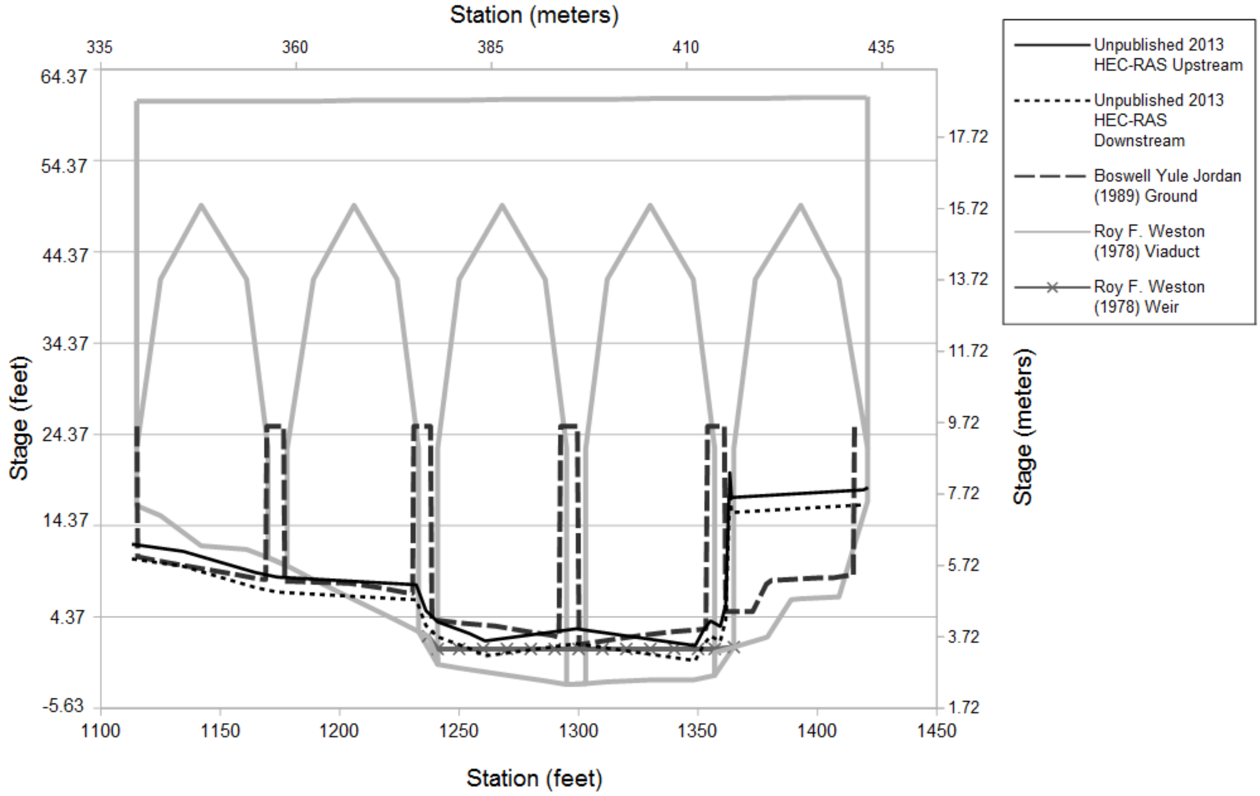

Figure 8. Approximations of Conestoga River cross section 6. Weir (solid gray line) from Roy F. Weston, Inc. (1978) is located at elevation 75.13 meters (246.5 feet) NGVD of 1929. East Walnut Street bypass added in 1990 is shown in the rightmost arch.

Attachment 2 (page 31)

REVISION COMMENTS

01576500 Conestoga River At Lancaster, PA.
Flood of June 23, 1972

The subject C/O measurement was reviewed, and revised, subsequent to receipt of complaint by PSC Engineers & Consultants, Inc. (8/4/89) in which it was pointed out that the computed discharge of 88,300 cfs was about 24,000 cfs greater than the 500-yr flood, but the 1972 profile was several feet below the 500-yr profiles of area Flood Insurance Studies at all points where 1972 HWM's were available. The step-backwater analyses for the FIS's are all consistent, being based on surveys and n-values of the SRBC. A plot of our surveyed 1972 HWM's (outside the C/O reach) on the Manheim Twp. FIS profiles indicated the 1972 profile was 2.5 to 2.7 ft higher than the FIS 100-yr profile, which was for the 35,610 cfs estimate from joint flood-frequency analysis of USGS and SRBC.

The first revision was made with the same fall in the surveyed reach (1.09 ft) as used in the original computation, but with adjustment of the approach section to a 'b' width further upstream--using the FIS-profile slope to obtain the fall adjustment for the revised approach location. This relocation was needed because the flow was fully eccentric, as evidenced by the previously unplotted HWM's at the right end of the bridge. Additionally, the skew was determined from the 7.5' topo. A discharge of <u>59,600</u> was determined for the first revision estimate.

A second revision was made because left-bank HWM's indicated a double peak, the second, and lower, peak being about 0.52 ft below the first. Approximate left-bank profiles for these peaks (plotted on original sheet 3) indicated a tailwater elevation of 71.6 ft for the higher tailwater profile. The indicated fall in the <u>surveyed</u> reach (0.75 ft) is considered the absolute maximum; concieveably, it could be as low as 0.60. Contracted-section parameters were recomputed on the basis of 71.60 ft for the tailwater, with n = .036. This roughness is based on available color slides, which indicate the tailwater roughness to equal or exceed that of the main channel at the approach. Approach roughness in sub-areas 1 & 2 probably should be higher than the values used, but minor revisions would have little impact on the computation.

The second, and final, C/O estimate is <u>50,300</u> cfs, which is considered "fair". The stage was revised to 27.90 ft on the basis of HWM's at the gage (outside) and comparison with the 1933 mark inside the water plant. When plotted, this estimate falls on a logical extension of the rating developed on the basis of measurements up to that of 1933 (no. 34A). This extension is consistent with the linear stage-area relation for the railroad bridge that forms the highwater hydraulic control.

H. N. Flippo, Jr.
5-23-90