16060000 South Fork Wailua River near Lihue, Kauai, Hawaii

(Gaging station, USGS Hawaii Water Science Center)

Review of peak discharge for the flood of April 15, 1963

Location: Lat 22°02’24”, long 159°22’58”, Hydrologic Unit 20070000, on right bank 0.2 mi upstream of Wailua Falls and 4.3 mi north of Lihue.

Published peak discharge: The published peak discharge for this flood is 87,300 ft^3/s and is rated poor.

Drainage area: 22.4 mi^2.

Data for storm causing flood: The April 15, 1963, flood on the South Fork Wailua River near Lihue, Kauai, Hawaii was one of several high flows in the Hawaiian Islands spawned by a series of storms that lasted from March to May 1963. Data for these storms are compiled and published in Vaudrey (1963). Rainfall ranged from about 8 to 18 in. over the drainage basin and probably averaged from about 10 to 15 in. (Rick Fontaine, USGS Hawaii Water Science Center, April 17, 2003, memorandum included in this flood file). The storms followed a year of drought conditions that ended in late December 1962 when a wetter than normal period started and extended through May 1963. Some areas of Kauai received in excess of 40 in. of rain in April 1963, which is not considered unusually high. There were no operational rain gages near the headwaters of the South Fork Wailua River on the south slope of Mount Waialeale.

The South Fork Wailua River near Lihue, Makaweli River near Waimea, and the Hanapepe River downstream of Manuali Stream near Elele are streamflow-gaging stations measuring runoff from the south slope of Mount Waialeale. High flows were reported on April 15, 1963, for all three sites but only the South Fork Wailua River site is on record as having extraordinary runoff. Some statistics for these sites for this storm are listed below.

Site	Drainage area (mi^2)	Discharge (ft^3/s)	Unit discharge [$(ft^3/s)/mi^2$]
South Fork Wailua River	22.4	87,300	3,900
Hanapepe River	18.8	39,000	2,070
Makaweli River	25	15,900	640

Historical photographs taken after the April 15, 1963, flood and during the 2003 review and described herein are provided in figures A240-A245.

Method of peak discharge determination: A two-section slope-area survey was conducted on May 10, 1963. Standard techniques were used to collect and analyze the field data. High-water marks were flagged 2 days after the flood at a reach extending several hundred feet upstream of Wailua Falls and starting immediately downstream of the bridge and road embankment that acts as a control for the stage-discharge relation at the gage site. Other sites were considered, but the reach was deemed the only practical place to conduct the survey. The field crew knew this was not a very good location because the reach was only long enough for two sections (A and B), and the hydraulic conditions were less than ideal.

The hydraulic conditions are less than ideal because of road overflow and a road embankment failure at the upstream end of the reach and a wide cross section B with assumed flow reversal (noncontributing flow) along the right bank. The magnitude of the presumed eddy is unknown, but the right-bank profile had almost no fall along the right bank from about half way between the sections to the end of the reach. Reverse flow possibly was present at the time of the peak discharge. The eddy effect probably was caused by channel geometry and debris caught on an old railroad bridge abutment just downstream of section B. The conveyance did not likely vary uniformly between sections. Hydraulic conditions are further questioned because of the very rapid rise and decline in stage. The stage rose 6 ft in a matter of a few minutes that followed a fairly significant decline in stage. This type of change often occurs when water is released from storage behind an obstruction upstream. The rapid onslaught of a very wet period following a very dry period could have triggered slope failures in the upstream part of the basin. Slope failures could have created a series of dams, but direct evidence for this possibility is not available.

The erratic left-bank high-water profile at the upstream end of the reach probably was caused by skewed flow over the road from a left-bank bypass channel and upstream channel alignment. This is the area where the road fill failed. The road embankment assumably failed just before the peak discharge, but the road more likely was immediately overwhelmed by the flood wave and failed after it became saturated and as the water receded. The flow is over the road every time the stage exceeds about a 12 ft gage height but evidently the embankment rarely fails. The road embankment failure probably created a large unsteady flood wave when the peak hit. This wave could account for the elevated and erratic high-water marks at the upstream end of the left bank. The origin and history of the left-bank bypass channel is unknown, but it looks like a natural channel. However, the bypass channel is directed toward the section where the road appears designed to fail to protect the bridge. There were few high-water marks available along the upstream right bank because of a cut bank.

Section B was subdivided, but a composite "*n*" value of 0.055 was used and may be low. Manning's "*n*," computed from the highest discharge measurements using the energy slope from the slope-area computation, ranged from 0.070 to 0.075. The bank vegetation was flattened by the high flow, and the peak discharge occurred so quickly that the downed trees and brush may be a remnant of flow duration rather than the peak discharge. Most of the bank vegetation could have survived the peak discharge and been flattened by debris pileup during the receding stage. Froude numbers were about 0.8, and velocity was about 20 ft/s.

Section B was subdivided in the original slope-area computation, but section A was not. During review, it was suggested that subdividing both sections and assigning independent subsection "*n*" values would yield a more accurate result. Harry Hulsing (USGS reviewer) calculated a discharge of 85,000 ft^3/s after subdividing section A confirming the original discharge. Hulsing used the roughness values assigned in the original computation for section B (0.045, main channel and left-bank composite, and 0.070 for the left-bank overflow). For the subdivision of section A, Hulsing used "*n*" values of 0.060 (left-bank overflow) and 0.045 (main channel and right-bank composite). There is no explanation of the roughness distribution used for section A in this computation. Rick Fontaine (USGS Hawaii Water Science Center) used the same approach in his 2003 analysis but used the field-estimated "*n*" values assigned by Ken Fowler (USGS) and weighted them by subsection area (as an approximation of weighting by subsection conveyance, which is the preferred weighting method). Fowler's distribution for section A is 0.055 (main channel and right-bank composite) and 0.070 (left-bank overflow). For section B, he used 0.035 (main channel and left-bank composite) and 0.120 (right-bank overflow). Fontaine's analysis resulted in a discharge of 68,800 ft^3/s.

The change in flow depths was so rapid that conditions may have violated the requirement for steady to gradually varied one-dimensional flow applicable to slope-area techniques. The stage increased more than 17 ft in 2 hours and decreased more than 5 ft in the 2 hours following the peak discharge. The road embankment could have failed any time during or after this period, further complicating the analysis. From the recorder chart, the first 6 ft of rise happened within a few minutes.

The recorded gage height for this peak discharge is verified by an inside high-water mark. No profile of outside high-water marks was surveyed past the gage, so it is unknown if the recorded peak discharge reflects the actual gage height. The alignment of the channel could cause a sloped water surface at the gage similar to the bank-to-bank discrepancy measured downstream. The road embankment breach assumably increased the peak stage by as much as 0.6 ft. Any increase in the rate of change in stage from the recorder trace is difficult to confirm. Peak stage should be verified by outside high-water marks for future peak discharges.

Possible sources of error: The assignment and distribution of roughness coefficients probably is the biggest source of error. Using a composite "*n*" value for subdivided sections with varied roughness is not recommended. A two-section solution does not provide any check on the computed discharge.

Flow could have been unsteady because of the rapid rise and decline in stage and the failure of the upstream road embankment. Section B had an unknown amount of noncontributing area. The conveyance did not vary uniformly between sections. At some time during or after the flood, the road embankment at the upstream end of the slope-area reach failed along the left bank, possibly releasing a surge of water and sediment.

Recommendations of what could have been done differently: A third section would have been beneficial. It would have been interesting to locate section A upstream about 40 ft and insert a third section about 60 ft downstream of section A at the break in slope on the right bank.

Subdividing the sections would yield a more accurate result. Using the field selected "*n*" values for each subsection would further refine the discharge estimate. The upstream part of the basin could have been inspected to look for possible landslide dams and their failure.

A critical depth cross section could be established at the head of Wailua Falls assuming flow is subcritical approaching the Falls. This may be a good assumption because Froude numbers of 0.8 were computed for the peak discharge. A high-water profile should have been surveyed past the gage to verify peak stage.

Site visit and review: The site was visited on February 25, 2003, by John Costa (USGS Office of Surface Water), Mike Nolan (USGS Western Region Surface-Water Specialist), Rick

Fontaine, Roy Taogoshi, and Clayton Yoshida (USGS Hawaii Water Science Center), and Gary Gallino (USGS retired). The field-review team inspected the cross-section locations, the road embankment, and a possible critical-depth section at the top of Wailua Falls. Assigned "*n*" values were discussed as was the possibility of landslide damming and failure in the upstream part of the basin.

Rick Fontaine and the USGS Hawaii Water Science Center subsequently investigated this flood peak. Several slope-area iterations were calculated. The most meaningful result came from use of the field-assigned "*n*" values listed as 'not used' in the field notes. He used the field-estimated "*n*" values and the cross-section subarea percentage applicable for each roughness coefficient. He subdivided both sections as suggested by Hulsing to avoid the questionable effect of computing alpha for a single cross section in this steep-gradient, fast-flowing reach. This computation produced a discharge of 68,800 ft^3/s. Fontaine also computed a flow estimate for culvert/road overflow at the road crossing that resulted in a discharge estimate of 47,000 ft^3/s. Slope-conveyance computations using historic high-flow cableway measurements produced a discharge of 64,100 ft^3/s. A flow estimate using an envelope curve and peaks of record for Kauai gaging stations produced a maximum likely peak discharge of 65,500 ft^3/s. A summary and explanation of these computations is included in Fontaine's April 17, 2003, memorandum.

Recommendation: The original peak discharge of 87,300 ft^3/s should be revised to 68,800 ft^3/s and the rating should remain "poor" because of the unknown effects of the road-embankment failure.

All analyses done by the USGS Hawaii Water Science Center suggest a smaller peak flow.

Rating comment: High-flow discharge measurements are made from a cableway at about the location of section A of the slope-area survey. The surface velocity during high flow is more than 15 ft/s on the surface, so no soundings are taken for the high-velocity subsections. Depths are obtained later after the stage falls and are accurate because of the bedrock stream bottom. Most high-flow measurements are made with a 75-lb weight which is inadequate for the depths and velocities experienced at this site. These measurements define a consistent stage-discharge relation but may not be the correct relation. A stay-line or a heavier weight would increase the accuracy of high-flow measurements and more accurately define the high end of the rating. It would allow a more standard measurement with 0.2 and 0.8-depth velocity data (0.6 is almost always less reliable than the two-point method). The upper end of the rating is defined by several high-flow measurements, and the extreme upper end is drawn through the 1963 slope-area discharge. The rate of change in discharge for the upper portion of the rating is 2,000 ft^3/s per 0.1 ft change in stage. This much increase seems extraordinary for a stream that is only about 300 ft wide. Discharge was 12,900 ft^3/s for measurement no. 336, with a gage height 16.60 ft. Discharge was 87,300 ft^3/s for the 1963 slope-area gage height of 22.9 ft. This means almost 75,000 ft^3/s had to flow through a cross section 6 ft deep and about 300 ft wide. The velocity would have to be more than 40 ft/s over the road embankment for this to be possible. This is considered unreasonable.

Figure A240. Reach looking downstream of streamgaging-station, South Fork Wailua River near Lihue, Kauai, 1963.

Figure A241. View upstream from cross-section 2, South Fork Wailua River near Lihue, Kauai, 2003.

Figure A242. View downstream at cross sections A and B, South Fork Wailua River near Lihue, Kauai, 2003.

Figure A243. View upstream at streamflow-gaging station, South Fork Wailua River near Lihue, Kauai, 2003.

Figure A244. Top of Wailua Falls waterfall downstream of streamflow-gaging station but upstream of slope-area reach, South Fork Wailua River near Lihue, Kauai, 2003.

Figure A245. Left bank floodplain at cross section 1, South Fork Wailua River near Lihue, Kauai, 2003.

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