Appendix A. Individual Evaluations of 30 Peak Discharges from 28 Extraordinary Floods in the United States

Seco Creek near D'Hanis, Texas

(Miscellaneous ungaged site in the Nueces River basin, USGS Texas Water Science Center)

Review of peak discharge for the flood of May 31, 1935

Location: This flood site is located at 29.4750 N and 99.3000 W about 11 mi north of D'Hanis, Tex.

Published peak discharge: The peak discharge for this site is 230,000 ft^3/s, as published in Crippen and Bue (1977). The rating is poor.

Drainage area: 142 mi^2.

Data for storm causing flood: Storm data could not be found for this flood. A large storm occurred in June 1935 that caused major floods on the West Nueces River as well as other streams in West Texas. The June 1935 storm was preceded by a major storm in May 1935 that caused a large peak discharge on Seco Creek. This is confirmed by several gaging-station records in the area that show a major peak discharge occurring on or about May 31, 1935. Historical photographs taken after the May 31, 1935, flood and during the 2003 review and described herein are provided in figures A1–A6.

Method of peak discharge determination: The peak discharge for this site is based on a two-section slope-area measurement. The high-water profile was defined for only one bank, and it is not clear if this was the left or right bank. The profile is uniform and fairly well defined, although there are no high-water marks within 60 ft of the downstream cross section. About 200 ft downstream of the downstream cross section, the profile seems to define a hydraulic jump of about 3 ft. The reach is straight. Roughness coefficients appear reasonable on the basis of the original photographs. Cross sections are too close together. About 90 percent of the total flow was in the main channel, with about 10 percent estimated in an overflow channel.

A slope-area computation (SAC) analysis was made for this review using the cross sections, water-surface elevations, and roughness coefficients as defined for the original computations. This SAC analysis attempted to duplicate the original computations as closely as possible. The reach was contracting. The SAC results indicated a discharge of 208,000 ft^3/s, for the main channel as compared to 209,000 ft^3/s for the original computations. Adding the estimated flow of 20,800 ft^3/s, the total peak discharge was computed as 229,000 ft^3/s (less than 1 percent less than the published peak discharge). Average main-channel cross-sectional area was 11,800 ft^2, average velocity was 17.6 ft/s, and Froude numbers ranged from 0.67 (upstream section) to 0.92 (downstream section).

Possible sources of error: The lack of a high-water profile for one bank is the most obvious source of error. In addition, the two cross sections are too close together, but the reach is contracting, which is a good feature. Otherwise, SAC computations confirm the original computations very closely, and the Froude numbers are reasonable. The 10 percent estimated overflow definitely is a possible source of error, and there is no way to verify this flow.

Recommendations of what could have been done differently: A high-water profile should have been obtained for both banks. This might have been difficult or impossible considering that there was overbank flow on one side, which may have made it difficult to find high-water marks. The field notes do not describe this very well.

Site visit and review: A field visit was made to the site on May 14, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS). The site was located using latitude and longitude with GPS. Physical markers were not available to locate cross sections. The site is described as being 11 mi upstream of D'Hanis, Tex.

The channel is straight and fairly wide and flat, composed of gravel, large cobbles, and small boulders. Scattered vegetation exists throughout the center part of the main channel, and both banks are heavily vegetated with mesquite and other trees. The main flow area is 200–300 ft wide in the lower part of the channel, with a top width of 600–800 ft. An attempt was made to locate the overflow area for which the flow was estimated. This area could not be located with any certainty, although a few low swales on the right side may have been the overflow area.

During the field visit, a local rancher (Mr. Rothe) drove up to inquire as to why we were there. He was the owner of the land adjacent to the indirect measurement site. Mr. Rothe stated that the flood of 1935 was the highest in his memory. He was in his teens at the time of the flood. It was interesting to note that a person named Rothe is listed in the field notes and assisted Tate Dalrymple in the 1935 survey. This person was the father of the current Mr. Rothe.

Recommendation: The original peak discharge of 230,000 ft^3/s should be accepted as published and rated as poor.

Any additional SAC analyses using different interpretations of the high-water profile would be pure speculation and would not be sufficient grounds for revising the original result.

Figure A1. View looking upstream of lower cross section, Seco Creek, Texas, June 8, 1935.

Figure A2. View looking upstream along right bank of lower cross section, Seco Creek, Texas, June 8, 1935.

Figure A3. View looking toward left bank at lower cross section, Seco Creek, Texas, June 8, 1935.

Figure A4. View from right to left bank near middle of slope-area reach, Seco Creek, Texas, June 8, 1935.

Figure A5. Slope-area reach of Seco Creek, Texas, looking upstream, June 2003.

Figure A6. Coarse materials in channel and flood plain of Seco Creek, Texas, June 2003. View toward right bank in slope-area reach. Notebook for scale.

08086150 North Fork Hubbard Creek near Albany, Texas

(Discontinued gaging station in the Brazos River basin, USGS Texas Water Science Center)

Review of peak discharge for the flood of August 4, 1978

Location: This flood is located at 32.7075 N and 99.2747 W, near Albany, Tex.

Published peak discharge: The published peak discharge for this discontinued gaging station is 103,000 ft^3/s. There are no published qualifications for this peak discharge; however, the USGS Water Science Center review by L.G. Stearns stated that it is of “fair reliability because of a scarcity of marks upstream and downstream.” The peak should be rated poor.

Drainage area: 39.4 mi^2.

Data for storm causing flood: Remnants of Tropical Storm Amelia dumped more than 29 in. of rainfall in Shackelford County causing flash flooding on Little Hubbard Creek. The storm set new records for 24-hour rainfall over 100- and 200-mi^2 areas. Six people were killed in Albany, Tex., and all roads into and out of the city were closed (Schroeder and others, 1987). Historical photographs taken after the August 4, 1978, flood and during the 2003 review and described herein are provided in figures A7–A41.

Method of peak discharge determination: The published peak discharge for this site is based on a combination of contracted-opening, culvert, and flow-over-road measurements. This indirect measurement was made at State Highway 6, which also is the location of the gaging station. All flow upstream of State Highway 6 was in one channel. The gaging station was washed out at a flow of about 2,050 ft^3/s on August 3, prior to the peak stage. The peak stage of 23.3 ft was determined from two poor high-water marks located about 200 ft downstream of the stream-gaging station.

Flow-over-road computations: The flow over the road was divided into two segments. The left overflow was about 1,600 ft wide, and the center and right overflow was about 3,200 ft wide. State Highway 6 goes through a large curve of about 45 degrees from one edge of the flood plain to the opposite edge. The general trend of the main channel and flood plain is nearly parallel to the highway on the left side and at a severe angle to the highway at the center and right side. The high-water profile upstream of the highway at the left overflow shows a drop of about 5 ft from the left to the right side, and the center and right overflow shows a drop of about 6 ft, for a total drop of at least 11 ft from the left side of the flood plain to the right side of the flood plain. All high-water marks (upstream and downstream) are 50 ft or more from the centerline of the highway. The high-water marks also are spaced far apart (as much as 600 ft) in places. These high-water marks probably were the only available marks, but reliability of the road-overflow results is questionable because of the distance between the high-water marks and the highway.

Road overflow computations were made assuming perpendicular flow, which appears to be a poor assumption considering the alignment of roadway and channel. Because high-water marks are not at the roadway (but rather 50 ft upstream), there also is uncertainty about friction losses between the upstream high-water marks and the crest of the roadway. The total discharge computed over the road was 76,820 ft^3/s, which is 75 percent of the overall total.

Bridge contracted-opening computations: Standard contracted opening procedures were used to compute flow through the bridge. However, the definition of the water-surface level, 23.3 ft, at the downstream side of the bridge is poor, based on only two high-water marks located about 200 ft downstream. The bridge was completely submerged, however, no flow was computed over the bridge because debris clogged the opening between the bridge deck and the handrail. The contraction coefficient also is questionable because it was computed as 1.00, which seems too high. Computed flow through the bridge opening was 20,500 ft^3/s.

Culvert flow computations: Standard culvert procedures were used to compute flow through the culvert on the right side of the flood plain. This resulted in a flow of 1,040 ft^3/s, which is a very small part of the total flow.

Possible sources of error: Sources of error primarily are related to the road-overflow computations and the contracted-opening measurement. The culvert computations are a very small part of the total discharge and are reasonable.

The left road overflow consists of a section of the highway that is nearly parallel to the main channel and flood plain. Although the original write-up states that flow was nearly perpendicular to this section of the road, this is difficult to believe, and there is no direct evidence to support this assumption. In fact, the high-water mark profile along the upstream and downstream sides of the highway would indicate otherwise. The water-surface profile parallel to the upstream side of the highway drops 5 ft, and along the downstream side of the highway, the water surface drops 6 ft. This large slope of the water surface parallel to the highway embankment would indicate (1) significant flow parallel to the highway, (2) probably very large angles of flow across the highway, and (3) uncertainty about the correct water-surface elevations to use for flow computations of the roadway subareas. Friction losses between the high-water marks and the crest of the highway is another possible factor that was not considered. The steep slope of the water-surface profile and the distance of 50–60 ft between the high-water marks and the highway most likely produced significant friction losses that were not accounted for in the computations.

The center and right road overflow is a long section of road overflow of about 3,200 ft, extending from about 1,350 ft left of the main channel bridge to about 1,850 ft right of the main channel bridge. This segment of road goes through a severe curve, and flow approaches it an angle, especially in the segment left of the main channel. Failure to consider the angle of approach in the road overflow computations is a possible source of significant error. In addition, failure to consider friction losses between the upstream high-water marks and the crest of the roadway also may be a possible source of error. The high-water profile along the upstream side of the road shows a drop of almost 6 ft, and the downstream profile shows a drop of about 7 ft, indicating significant flow and velocity parallel to the highway.

The contracted opening computations also are a possible source of error. A review of these computations indicates a number of mark-overs and corrections that are difficult to follow. There also are a few misinterpretations of the procedure defined by Matthai (1967). The contracted-opening computations are difficult because the bridge was completely submerged, including the bridge deck and handrails. The method is not well defined for such conditions. The most obvious errors are:

- A math error is in the computation of the contraction coefficient *m*. The value should be 0.10 and not 0.19.
- The contracted area, A_3, was not correctly interpreted. The computations use the net area rather than the gross area as defined by Matthai (1967). Again, this is not an easy interpretation because of the completely submerged bridge, and Matthai (1967) is not entirely clear for this type of contraction.
- The value of y_3 is questionable, depending on the value of A_3.
- The wetted perimeter of the contracted section was incorrectly computed. The computation should include the lower chord of the bridge.
- The downstream water-surface elevation is questionable because it is based on two high-water marks, rated poor, located about 200 ft downstream. A third high-water mark, also rated poor, that was more than 1 ft higher and located in the same vicinity was not used.

Recommendations of what could have been done differently: A different approach is difficult to recommend because of the extreme magnitude of this flood. A two-dimensional method is mentioned in the main body of this report; however, two-dimensional models were still in their infancy in 1978 and probably would not have been very useful. A slope-area survey might have been possible in the reach downstream from the gaging station. The flood plain is about 0.6 mi wide, but the reach appears straight, and a two-section slope-area measurement might have been less questionable than the road-overflow and contracted-opening measurement. A three-section slope-area measurement likely could not have been made because the reach would not have been long enough.

Finally, any evidence of direction of flow, both upstream and downstream of the highway, could have been defined and documented. However, field evidence is still questionable because of uncertainty if the field evidence represents flow at the peak or flow at a lower stage of the recession. Some additional high-water marks downstream of the bridge would have been helpful in evaluating fall through the bridge and in defining the correct stage for this flood.

Site visit and review: A field visit was made to the site on May 12, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS). The field inspection reinforced the suspicion that the angle of approach for road overflow may have been significant and that the computed discharge probably is too high. The roadway has been altered (rebuilt) since 1978, probably just re-surfacing. A few roadway elevations were checked by levels and were determined to be slightly higher than those surveyed for the indirect measurement. This flood obviously is a two-dimensional flow problem and probably can not be computed with much accuracy using one-dimensional methods.

In the process of this review, some broad assumptions were made to evaluate the effects of angle of flow on the road-overflow computations. For the left overflow, an approach angle of 60 degrees was assumed, which results in a correction factor of 0.5 (cosine of 60 degrees). Applying this correction, the discharge of the left overflow would be:

$$0.5 \times 4{,}670 = 2{,}335 \text{ (rounded to } 2{,}340 \text{ ft}^3\text{/s).}$$

The center and right overflow was divided into two sections, with angle corrections applied as follows:

Station 45 to 786, angle = 60 degrees (cosine=0.5), $Q = 7{,}856 \times 0.5 = 3{,}930$ ft^3/s;

Station 786 to 3,245, angle = 30 degrees (cosine=0.866), $Q = 68{,}968 \times 0.866 = 59{,}730$ ft^3/s;

Angle corrected $Q = 3{,}930 + 59{,}730 = 63{,}700$ ft^3/s.

Total road overflow, corrected for assumed angles $= 2{,}340 + 63{,}660 = 65{,}990$ ft^3/s.

Friction losses between the high-water marks and the roadway were not accounted for because there is insufficient information to make even an estimate. Friction losses would further reduce road overflow, but the magnitude of this reduction is difficult to estimate.

A recomputation of flow through the bridge opening, using the corrections previously noted, resulted in a discharge of 22,500 ft^3/s. This is 2,000 ft^3/s greater than the original value of 20,500 ft^3/s.

The following summarizes the results of the recomputations:

Location	Original computation (ft^3/s)	Recomputed (ft^3/s)
Left overflow	4,670	2,340
Right overflow	76,820	63,660
Bridge	20,500	22,500
Culvert	1,040	1,040
Total	103,030	89,540

A second method of recomputation is based on the slope-conveyance method. The approach section for the highway, bridge, and culvert that is included in the original computations appears to be fairly representative of the complete valley. The approach section is at a general angle of about 28 degrees to the main channel and flood plain. Cross-section properties at 1-ft intervals were determined using the USGS slope-area computation (SAC) program and adjusted by the cosine of 28 degrees. The adjusted conveyance determined in this manner was used for the slope-conveyance computations.

Channel slope was estimated using three methods. First, the channel slope was estimated from contour intervals on the topographic map to be 0.0035. Second, the slope was estimated from the 1978 high-water profile defined along the downstream side of the left highway embankment to be 0.0028. The 1978 high-water profile is approximately parallel to the left edge of the flood plain in this reach. Third, slope was computed using the slope-conveyance method from rating-curve discharges for stages from 5 to 21 ft. The upper end of the rating is questionable, so the higher values of the rating were not given much weight. All slopes determined by these three methods were plotted against stage. On the basis of this graph, a slope of 0.0020 was used as the best estimate for all stages above 16 ft. This slope was merged with a smooth transition curve to the rating curve slopes below 15 ft.

This “best” estimate of the relation between stage and slope was used to compute rating-curve plotting points. The discharge for the August 1978 flood (stage = 23.3 ft) was determined to be 58,600 ft^3/s using this method.

Recommendation: The original peak discharge of 103,000 ft^3/s, as originally computed, should be revised.

This peak discharge is based on flow assumptions regarding road overflow that are not correct and that can not be reasonably evaluated using one-dimensional methods. The peak discharge, as determined by two independent recomputations ranged from 58,600 to 89,500 ft^3/s. The mean of these two values is 74,000 ft^3/s, which probably is a more reasonable value to use. If used, this revised peak discharge should be rated as poor, with a probable error of ±20 percent. The unit runoff, based on 74,000 ft^3/s, is 1,878 $(ft^3/s)/mi^2$.

For comparison, 1978 flood peaks in this area are:

- Hubbard Creek below Albany, Tex. Drainage area = 613 mi^2. Peak discharge = 330,000 ft^3/s, unit discharge = 538 ft^3/s.
- Deep Creek at Moran, Tex. Drainage area = 228 mi^2. Peak discharge = 13,000 ft^3/s, unit discharge = 57 ft^3/s. This site is about 15 mi southeast of Albany.
- North Fork Hubbard Creek near Albany, Tex. (this review site; map no. 2, fig. 1). Drainage area = 39.4 mi^2. Published peak discharge = 103,000 ft^3/s, unit discharge = 2,610 ft^3/s.

Figure A7. View from right end of bridge looking across bridge, North Fork Hubbard Creek near Albany, Texas, August 5, 1978. Flood of August 4, 1978, was about 2 feet over handrail.

Figure A8 View of left abutment and upstream wingwall, North Fork Hubbard Creek near Albany, Texas, September 20, 1978.

Figure A9. View of right abutment and upstream wingwall, North Fork Hubbard Creek near Albany, Texas, September 20, 1978.

Figure A10. View looking downstream side of bridge from left bank, North Fork Hubbard Creek near Albany, Texas, August. 4, 1978.

Figure A11. View from near left end of bridge at gaging station looking downstream, North Fork Hubbard Creek near Albany, Texas, August 5, 1978.

Figure A12. View of downstream side of bridge from right bank, gage shelter in center of stream, North Fork Hubbard Creek near Albany, Texas, August 4, 1978.

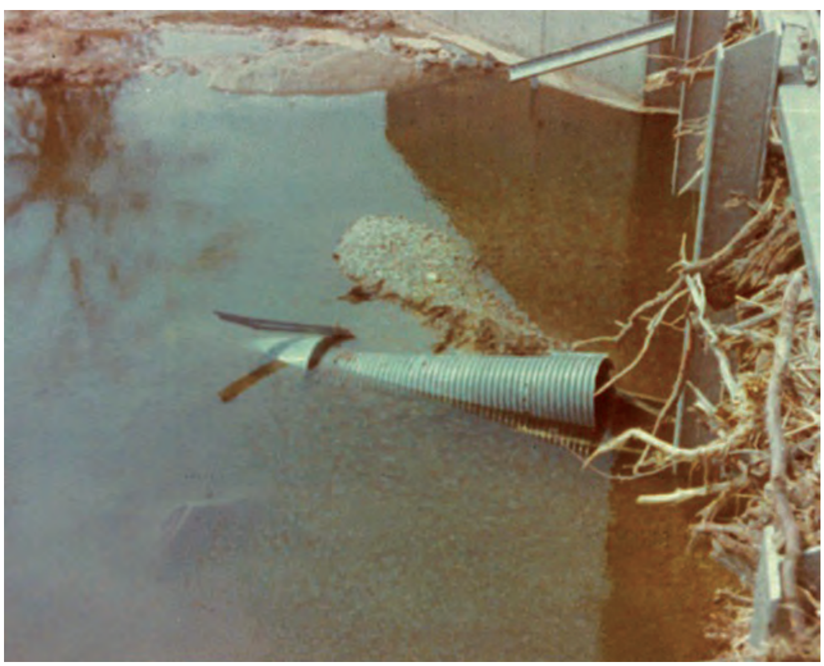

Figure A13. View of gage shelter and well taken from top of bridge, North Fork Hubbard Creek near Albany, Texas, August 10, 1978.

Figure A14. View of bridge and channel downstream from left bank, shelter and well being lifted out of stream, North Fork Hubbard Creek near Albany, Texas, August 10, 1978.

Figure A15. View of motel on right bank downstream of U.S. Highway 180 bridge at Albany, Texas. Two new cars forced into motel by water, North Fork Hubbard Creek near Albany, Texas, August 4, 1978.

Figure A16. Upstream side of State Highway 6 overflow near right bank looking toward left bank with rod held near fence at the high-water mark in the approach section, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A17. View from 50 feet upstream of State Highway 6 near left bank of main channel looking downstream with rod near high-water mark on left bank downstream of highway, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A18. View near crest of State Highway 6 looking upstream with rod at high-water mark at approach section near left bank of main-channel overflow, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A19. View from 200 feet to left of bridge and 25 feet downstream of State Highway 6 looking upstream across highway at approach section with rod held at high-water mark, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A20. View from about 200 feet to left and 25 feet downstream of gaging station at bridge looking right and across at approach section and right bank. Rod held near high-water mark in approach section, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A21. View from near gaging station at bridge looking downstream at channel, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A22. View from near gaging station on upstream side of bridge looking upstream, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A23. View at approach section to bridge at gaging station looking downstream at bridge and channel, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A24. View from about 100 feet downstream of culvert located to right of gaging station looking upstream at culvert and approach section with rod held on upstream side of culvert near high-water mark in approach section, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A25. View at approach section of culvert looking downstream at culvert. Note flag in tree just above rod for elevation downstream of highway, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A26. View about 100 feet to left of culvert on downstream shoulder of State Highway 6 looking across culvert at right bank and approach section. Rod held at high-water mark in approach, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A27. View from 50 feet to right of culvert on downstream shoulder looking across culvert with rod held at high-water mark in approach section, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A28. View about 100 feet to right of culvert on upstream shoulder looking slightly downstream and across to left bank. Rod held in approach section at high-water mark. The first string of trees is the main channel, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A29. View from 25 feet downstream of State Highway 6 near left end of overflow looking upstream at approach section. Main channel in background is North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A30. View from near left-bank overflow section looking to right bank showing State Highway 6 and approach section, North Fork Hubbard Creek near Albany, Texas, August 15, 1978.

Figure A31. View looking downstream of bridge, North Fork Hubbard Creek near Albany, Texas, May 13, 2003.

Figure A32. View looking upstream of bridge, North Fork Hubbard Creek near Albany, Texas, May 13, 2003.

Figure A33. View looking across State Highway 6 and culvert toward left bank of flood plain, North Fork Hubbard Creek near Albany, Texas, May 13, 2003.

Figure A34. View from upstream side of State Highway 6 bridge, North Fork Hubbard Creek, near Albany, Texas, May 12, 2003.

Figure A35. View looking upstream at main channel from State Highway 6 bridge, North Fork Hubbard Creek, near Albany, Texas, May 12, 2003.

Figure A36. View looking left to right from State Highway 6 main-channel bridge, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Figure A37. View looking upstream of State Highway 6 bridge, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Figure A38. View looking downstream of State Highway 6 bridge, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Figure A39. View looking right to left of State Highway 6 bridge, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Figure A40. View looking upstream of downstream side of road near left end of bridge, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Figure A41. View looking left to right from curve in road, North Fork Hubbard Creek near Albany, Texas, May 12, 2003.

Mailtrail Creek near Loma Alta, Texas

(Miscellaneous ungaged site in the Rio Grande River basin, USGS Texas Water Science Center)

Review of peak discharge for the flood of June 24, 1948

Location: This flood was located about 43 mi north of Del Rio, Tex., at 29.9792 N and 100.7375 W.

Published peak discharge: The peak discharge for this site, as published in Asquith and Slade (1995), is 170,000 ft^3/s. The rating is poor.

Drainage area: 75.3 mi^2.

Data for storm causing flood: The following information is given in the original field notes for this flood:

> "John Galloway, who has lived for 45 years on the right bank about ½ mile above slope reach, states that this flood was 2 to 3 feet higher than known before; the previous maximum stage occurring in 1932. He measured 22"+ of rainfall at his ranch, with rain beginning in morning and continuing about 12 hours. He stated there were 3 peaks, the highest coming at about 10 a.m."

Historical photographs taken after the June 24, 1948, flood and during the 2003 review and described herein are provided in figures A42–A58.

Method of peak discharge determination: The peak discharge for this site is based on a three-section slope-area measurement. All flow was in one channel. High-water profiles were defined on both banks, although the right-bank profile is poorly defined with only a few high-water marks. The left-bank profile is well defined. The reach is straight. The original computations attempt to use all three cross sections, but the final result of 170,000 ft^3/s is based on only the middle (section 2) and downstream (section 3) cross sections. Roughness coefficients appear reasonable based on the original photographs.

Two separate slope-area computation (SAC) analyses were conducted for this review. The first SAC analysis used all three cross sections, with cross-section subdivisions and roughness coefficients used as in the original computations. Water-surface elevations for the cross sections also were the same as the original computations. This SAC analysis attempted to duplicate the original computations as closely as possible. The reach is contracting throughout. The SAC peak discharge using all three cross sections is 175,000 ft^3/s (3 percent higher than the published peak discharge). The SAC peak discharge, based on sections 2 to 3, was 168,000 ft^3/s (1 percent less than the published peak discharge). Average cross-sectional area is 13,200 ft^2, average velocity is 13.4 ft/s, and Froude numbers ranged from 0.61 (section 1), to 0.74 (section 2), and 0.98 (section 3).

The second SAC analysis used all three cross sections; however, subdivisions of cross sections were somewhat different than the original, and roughness coefficients were slightly different to conform to the different subdivisions. Water-surface elevations for the cross sections were the same as the original computations. This second SAC analysis yielded peak discharges of 172,000 ft^3/s for a three-section computation and 169,000 ft^3/s for a two-section (sections 2 and 3) computation. The reach is contracting, and cross-section properties are similar to those for the first SAC analysis.

Possible sources of error: The water-surface profile probably is the primary source of error. Other interpretations of the water-surface elevations at each cross section could be made. The lack of good high-water profile definition on the left bank is the primary uncertainty. Cross-section subdivision in the original computations was not done exactly as currently practiced by the USGS; however, the method used did not introduce any significant error. In addition, cross sections are too close together, but the reach is contracting throughout, which is a good feature. Froude numbers are reasonable.

Recommendations of what could have been done differently: This is a good slope-area measurement made under poor field conditions, and it is doubtful that anything could have been done to improve the results.

Site visit and review: A field visit was made to the site on May 13, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS). The site was located using latitude and longitude with GPS. Physical markers were not available to locate cross sections. The site is described as being 1.0 mi upstream of U.S. Highway 277 and 0.5 mi downstream of the Galloway Ranch house. The ranch house was located on the basis of conversations with local ranchers.

The channel appears to be much more densely vegetated in 2003 than it was in 1948 based on photographs taken when the slope-area measurement was made. There is no clearly defined main channel but rather a wide flood plain consisting of gravel, large cobbles, and small boulders. The main flow area is about 1,000 ft wide. A small bench or overflow area is indicated on the left side by the cross sections surveyed in 1948. The right side has steep banks. In 2003, a rather dense growth of scrub mesquite was observed throughout the site.

An interview with local ranchers indicated that the flood of 1948 rose quickly, giving no time to evacuate livestock from the flood area. Many sheep, goats, cattle, and horses were swept away by the flood. Ranch equipment of various sorts also was washed away. Velocities were very high, and the water surface had what the ranchers described as large waves.

<u>**Recommendation**</u>: The original peak discharge of 170,000 ft^3/s should be accepted as published and rated poor.

Figure A42. View looking downstream at downstream cross section from near center of channel, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A43. View looking downstream at downstream cross section from near right bank, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A44. View looking downstream of downstream cross section to right of center of main channel, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A45. View looking across channel from right bank near downstream cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A46. View looking downstream from center channel at upstream cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A47. View looking downstream from right bank at upstream cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A48. View looking downstream from near center of channel at upstream cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A49. View looking downstream from left side of main channel at upstream cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A50. View looking downstream from right bank at middle cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A51. View looking downstream at middle cross section–left-bank overflow, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A52. View looking downstream from near center of channel at middle cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A53. View looking downstream from near center of main channel at middle cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A54. View looking downstream from left side of main channel at middle cross section, Mailtrail Creek near Loma Alta, Texas, flood of June 24, 1948.

Figure A55. View looking upstream from channel in slope-area reach, Mailtrail Creek near Loma Alta, Texas, May 13, 2003.

Figure A56. View looking upstream near upstream cross section of slope-area reach, Mailtrail Creek near Loma Alta, Texas, May 13, 2003.

Figure A57. View looking downstream from slope-area reach, Mailtrail Creek near Loma Alta, Texas, May 13, 2003.

Figure A58. View looking downstream from U.S. Highway 277 bridge below slope-area reach, Mailtrail Creek near Loma Alta, Texas, May 13, 2003.

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West Fork Nueces River near Kickapoo Springs, Texas

(Miscellaneous ungaged site in the Nueces River basin, USGS Texas Water Science Center)

Note: This site was originally named “33 miles above Brackettville,” later changed to “28 miles above Brackettville,” and on some documents just “near Brackettville.” The current name “near Kickapoo Springs” was assigned at some later date. The measurement site is officially described as “33 miles above the gage near Brackettville.”

Review of peak discharge for the flood of June 14, 1935

Location: This flood was located about 33 mi north of Bracketville, Tex., at 29.7583 N and 100.3958 W.

Published peak discharge: The peak discharge for this miscellaneous site, as published in Asquith and Slade (1995), is 580,000 ft^3/s. The rating is poor.

Drainage area: 402 mi^2.

Data for storm causing flood: Very little information is available for the June 1935 storm in the Nueces River basin. USGS National Water Summary (Paulson and others, 1991) has a short narrative for the South Llano and James River basins, which are just north of the Nueces River basin. Paulson and others (1991) indicate that intense rainfall of more than 18 in. fell during June 9–15, 1935, in the South Llano and James River basins that created record floods at several points in these basins. Otherwise, no information could be found for rainfall in the Nueces River basin. Historical photographs taken after the June 14, 1935, flood and during the 2003 review and described herein are provided in figures A59–A65.

Method of peak discharge determination: The peak discharge for this site is based on a two-section slope-area computation. All flow was in one channel. The original survey defined two cross sections (sections 1 and 3) that were 568 ft apart. Almost 2 years after the original survey, two additional cross sections (2 and 4) were surveyed to confirm the cross-sectional areas. One additional section (section 2) was located between sections 1 and 3, and another section (section 4) was located downstream of section 3. These additional sections apparently were not used to compute the peak discharge because there is no record of them in the files. No additional high-water marks could be found during the second survey.

High-water profiles were defined on both banks, although the right-bank profile is subject to considerable interpretation. Marks on the right bank show large differences (as much as 5 and 6 ft) in the upstream end of the reach. There may have been two or more peak discharges, or there may have been large waves near the right bank. The analyst of the original computations used the upstream high-water marks to define the high-water profiles. The left-bank profile is well defined, but is about 3 ft lower than the right-bank profile.

Roughness coefficients appear reasonable. The field-assigned Manning’s roughness coefficient of 0.030 was used throughout the reach. Jarrett’s (1994) equation computes a coefficient of 0.025 for section 3 and 0.029 for section 1.

The original files indicate that at least four different computations were used based on different water-surface slopes. Peak discharges ranged from 537,000 to 612,000 ft^3/s. For this review, three separate slope-area computation (SAC) analyses were done. The first analysis used the two original cross sections, and the upstream profile on the right bank. Water-surface elevations were nearly the same as in the original computations. This SAC analysis attempted to duplicate the original computations as closely as possible. A peak discharge of 522,000 ft^3/s was computed. The reach is contracting throughout with Froude numbers of 0.58 (upper) and 0.63 (lower).

The second SAC analysis was the same as the first, except three cross sections were used. The additional cross section (section 2), was inserted. Again, the upstream profile on the right bank was used. The SAC peak discharge, based on three sections, was 523,000 ft^3/s, contracting throughout. Froude numbers ranged from 0.58 (section 1), to 0.60 (section 2), and 0.63 (section 3).

The third SAC analysis used the same three cross sections; however, in this case, the downstream profile on the right bank was used. The third analysis yielded a peak discharge of 486,000 ft^3/s for a three-section computation, contracting throughout, and Froude numbers similar to those from the second SAC analysis.

Possible sources of error: The West Fork Nueces River near Kickapoo Springs, Tex., seems to be a good slope-area site; however, the lack of good high-water profile definition on the right bank is the primary uncertainty in this poor measurement. In addition, the cross sections are too close together, but the reach is contracting throughout, which is a good feature. Froude numbers are reasonable. The most likely source of error for this site is in the interpretation of the high-water profile.

Recommendation of what could have been done differently: A longer reach would have been better, and this was attempted about 2 years after the flood, but high-water marks could not be defined at that time. This measurement received thorough review, including a review by the USGS Chief Hydraulic Engineer in Washington, D.C. On the basis of his review, additional cross sections were surveyed; however, this did not result in a change to the original computed discharge.

Site visit and review: A field visit was made to the site on May 13, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS). The site was located using latitude and longitude with GPS. Physical markers were not available to locate cross sections.

The channel is about 600 ft wide, relatively flat, and open. It is composed of gravel, large cobbles, and small boulders. Both banks are fairly steep. It appears to be a very good slope-area site, but the measurement is poor.

Recommendation: The original peak discharge of 580,000 ft^3/s should be accepted as published.

The three SAC analyses indicate that the peak discharge is about 10 to 16 percent less than the published peak discharge. This is based on the original interpretations and on reviewers interpretations of the data. In light of the uncertainties in water-surface profiles, the difference is not considered large enough to warrant a revision to the original published peak discharge.

Figure A59. View looking across stream from right bank at upstream cross section, West Fork Nueces River near Kickapoo Springs, Texas, June 1935. Slope-area section for flood of June 14, 1935.

Figure A60. View looking across and upstream of downstream cross section, West Fork Nueces River near Kickapoo Springs, Texas, June 1935.

Figure A61. View looking upstream of downstream cross section, West Fork Nueces River near Kickapoo Springs, Texas, June 1935.

Figure A62. View looking upstream toward slope-area reach, West Fork Nueces River near Kickapoo Springs, Texas, June 13, 2003.

Figure A63. View looking from right to left bank in middle of slope-area reach, West Fork Nueces River near Kickapoo Springs, Texas, June 13, 2003.

Figure A64. View looking upstream near middle of slope-area reach, West Fork Nueces River near Kickapoo Springs, Texas, June 13, 2003.

Figure A65. View looking downstream from right bank looking from middle of slope-area reach, West Fork Nueces River near Kickapoo Springs, Texas, June 13, 2003.

08190500 West Fork Nueces River near Brackettville, Texas

(Gaging station in the Nueces River basin, USGS Texas Water Science Center)

Review of peak discharge for the flood of June 14, 1935

Location: This flood was located about 15 mi northeast of Bracketville, Tex., at 29.4725 N and 100.2361 W.

Published peak discharge: The peak discharge for this gaging station is 550,000 ft^3/s, as published in the gaging-station Peak-Flow File, as a historical peak. The stream-gaging station did not exist in 1935. No rating is given to this extrapolated flood discharge.

Drainage area: 694 mi^2.

Data for storm causing flood: Very little information is available for the June 1935 storm in the Nueces River basin. Paulson and others (1991) has a short narrative for the South Llano and James River basins, which are just north of the Nueces River basin. That report indicates that intense rainfall of more than 18 in. fell during June 9–15, 1935, in the South Llano and James River basins that created record floods at several points in these basins. Other information could not be found for rainfall in the Nueces River basin. Photographs taken during the 2003 review and described herein are provided in figures A66–A68.

Method of peak discharge determination: The published peak discharge for this site is based on drainage-area interpolation using base-10 logarithms of the peak discharges on the West Fork Nueces River near Cline, 24 mi downstream, and near Kickapoo Springs, 33 mi upstream. The peak discharges for the Cline and Kickapoo Springs sites were determined by slope-area measurements that have been reviewed as a part of this study. Data for these sites are as follows:

Kickapoo Springs
Drainage area = 402 mi^2 Q = 580,000 ft^3/s (slope-area computation [SAC] program)

Brackettville (gage)
Drainage area = 694 mi^2 Q = 549,000 ft^3/s (interpolation)

Cline
Drainage area = 880 mi^2 Q = 536,000 ft^3/s (SAC program)

The precise interpolation gives a peak discharge of 549,000 ft^3/s. This apparently was rounded to 550,000 ft^3/s, which seems reasonable.

The published peak discharge for the gaging station also is confirmed with good agreement from the high-water rating for the gaging station. The peak stage at the gaging station for the 1935 flood is published in the Peak Flow File as 40.00 ft and is based on floodmarks for the 1955 flood and local resident information. Considering the method used to determine this stage, it would be better to publish this as 40.0 ft or possibly 40 ft. The method used to determine the peak stage is summarized below.

> 1935 flood, stage = 48.0 ft at a site 0.6 mi upstream (two high-water marks pointed out by local resident Mr. L.E. Bruce, in 1955).
>
> 1935 flood, stage estimated as 48.0 – 8.1 = 39.9 ft (rounded to 40.0 ft).
>
> 1955 flood, stage = 35.2 ft at a site 0.6 mi upstream, from floodmarks.
>
> 1955 flood, stage = 27.1 ft at gaging station.
> Difference 35.2 – 27.1 = 8.1 ft (fall in reach 0.6 mi).

Possible sources of error: Interpolation methods of this type are subject to uncertainties, especially in a reach of about 60 mi and with a drainage-area increase of 100 percent. It is unlikely that the peak traveled for such a distance without considerable attenuation. Therefore, there must have been significant contributions from tributaries along the reach to sustain the peak discharge at a level of more than 500,000 ft^3/s. The rating curve is the best confirmation provided the peak stage at the gaging station is accurate.

Recommendations of what could have been done differently: Considering that the gaging station was not established until 5 years after the peak discharge occurred, the two methods used probably were the best that could be done.

Site visit and review: A field visit was made to the gaging station on May 13, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS).

Recommendation: The original peak discharge of 550,000 ft^3/s should be accepted as published.

Considering that the peak discharge is confirmed by two independent methods of computation, it can be published without qualification. The peak stage should be published as "about 40 ft or about 40.0 ft." The use of hundredths of a foot is not warranted.

Figure A66. View looking from left to right bank, West Nueces River near Brackettville, Texas, May 13, 2003.

Figure A67. View looking downstream of streamflow-gaging station, West Nueces River near Brackettville, Texas, May 13, 2003.

Figure A68. View from right to left bank at streamflow-gaging station, West Nueces River near Brackettville, Texas, May 13, 2003.

West Fork Nueces River near Cline, Texas

(Miscellaneous ungaged site in the Nueces River basin, USGS Texas Water Science Center)

Note: This site was originally named “8 miles above Cline,” later changed to “24 miles downstream from gage near Brackettville,” and on some documents just “near Brackettville.” The current publication name “near Cline” was assigned at some later date. The measurement site is officially described as 24 mi downstream from gage near Brackettville.

Review of peak discharge for the flood of June 14, 1935

Location: The flood was located about 18 mi east of Bracketville, Tex., at 29.3383 N and 100.1102 W.

Published peak discharge: The peak discharge for this miscellaneous site, as published in Asquith and Slade (1995), is 536,000 ft^3/s. The rating is fair.

It is important to note that this site is not one of the 28 extraordinary floods reviewed in this report. However, it is necessary to review this measurement because it is used in conjunction with the measurement at Kickapoo Springs, Tex., to define the peak discharge at gaging station 08190500, West Nueces River near Brackettville, Tex., for the June 14, 1935, flood. The gaging-station peak discharge is one of the 30 peak discharges selected for this review.

Drainage area: 880 mi^2.

Data for storm causing flood: Very little information is available for the June 1935 storm in the Nueces River basin. Paulson and others (1991) has a short narrative for the South Llano and James River basins, which are just north of the Nueces River basin. Paulson and others (1991) indicate that intense rainfall of more than 18 in. fell during June 9–15, 1935, in the South Llano and James River basins that created record floods at several points in these basins. Other information could not be found for rainfall in the Nueces River basin. Historical photographs taken after the June 14, 1935, flood and during the 2003 review and described herein are provided in figures A69–A75.

Method of peak discharge determination: The peak discharge for this site is based on a two-section slope-area computation. All flow was in one channel. High-water profiles were defined on both banks, although the two profiles are quite different— the left-bank profile is considerably steeper than the right-bank profile and the right-bank profile is well defined with many high-water marks. The right-bank profile indicates the possibility of standing waves, whereas the left-bank profile does not have many high-water marks, and there is a fairly large scatter of the marks in the downstream end of the reach. The slope defined by the left-bank profile is twice the slope defined by the right-bank profile. The analyst of the original computations used the upstream high-water marks for the left-bank profile and averaged the high-water marks on the right bank.

The original computations used a roughness coefficient of 0.04 for both cross sections with no subdivision. This computation was a simple application of Manning’s equation and used the average slope defined by the high-water profiles. Corrections were not made for velocity head differences, although differences would have been small because the two cross sections were nearly the same with a slightly contracting reach. The average cross-sectional area used in the original computations was 33,900 ft^2. Average velocity in the reach was 15.6 ft/s.

For this review, two separate slope-area computation (SAC) analyses were conducted. The first analysis used the original two cross sections and the same profiles used in the original computations in an attempt to duplicate the original computations. A peak discharge of 518,000 ft^3/s was computed. The reach is slightly contracting with Froude numbers of 0.59 (upper) and 0.61 (lower).

The second SAC analysis used subdivided cross sections and variable roughness coefficients. The cross sections were subdivided primarily on the basis of shape, with roughness coefficients assigned on the basis of the field-note descriptions and the photographs. The same water-surface elevations were used as in the first SAC analysis. The peak discharge was computed as 509,000 ft^3/s. Area, velocity, and Froude numbers were similar to those from the first SAC analysis.

On the basis of the two SAC analyses, the original computed discharge may be about 3 to 5 percent too high. However, this difference can be accounted for by different interpretations of the left-bank high-water profile and slightly different roughness coefficients. A significant shortcoming of this measurement is that the reach is too short. The channel is about 1,700 ft wide, and the distance between cross sections is only 700 ft. The fall in the reach is 2.25 ft.

Possible sources of error: The interpretation of the high-water profiles and the fact that one bank indicates a much steeper slope than the other are the most likely sources of error. The shortness of the reach is another possible source of error. Froude numbers are small considering the magnitude of this flood.

Figure A69. View looking across and upstream towards left bank from downstream cross section, West Nueces River 8 mi upstream of Cline, Texas. June 1935.

Recommendations of what could have been done differently: A longer reach with an additional cross section would have been appropriate.

Site visit and review: A field visit was made to the site on May 14, 2003, by John Costa (USGS Office of Surface Water), John England (Bureau of Reclamation), and Vernon Sauer and Raymond Slade (USGS). The site was located using latitude and longitude with GPS. Physical markers were not available to locate cross sections.

The main channel is relatively flat and open. The streambed consists of gravel, large cobbles, and small boulders. Both banks have a fairly dense growth of small trees and brush.

Possible sources of error: This seems to be a good slope-area measurement site; however, the uncertainty of the left-bank profile and the fact that one bank indicates a much steeper slope than the other are the main possibilities of error. Another problem is that the two cross sections are too close together, but the reach is uniform and slightly contracting, which is a good feature. Froude numbers are reasonable.

Recommendation: The original peak discharge of 536,000 ft^3/s should be accepted as published.

Figure A70. View looking at West Nueces River 8 mi upstream of Cline, Texas, June 1935.

Figure A71. View looking upstream of downstream cross section at station 1, West Nueces River 8 mi upstream of Cline, Texas, June 1935.

Figure A72. View looking toward left bank and downstream of upstream cross section, West Nueces River 8 mi upstream of Cline, Texas, June 1935.

Figure A73. Coarse bed material in slope-area reach of West Nueces River 8 mi upstream of Cline, Texas, June 2003.

Figure A74. View looking upstream of slope-area reach, West Nueces River 8 mi upstream of Cline, Texas, June 2003.

Figure A75. View looking downstream of slope-area reach, West Nueces River 8 mi upstream of Cline, Texas, June 2003.

Jimmy Camp Creek at Fountain, Colorado

(Miscellaneous ungaged site, Arkansas River basin, USGS Colorado Water Science Center)
(1976–present, streamflow-gaging station number 07105900)

Review of peak discharge for the flood of June 17, 1965

Location: This flood was located about 3.5 mi north-west of Fountain, Colo., at 38.7196 N and 104.6459 W.

Published peak discharge: The peak discharge, as published in 1965, is 124,000 ft^3/s, June 17, 1965. The original measurement was rated fair; but this report recommends that the rating be downgraded to poor.

Drainage area: 54.3 mi^2. Map scale used for defining the drainage area is unknown. Current gaging station drainage area is 65.6 mi^2.

Data for storm causing flood: The flood of 1965 was the result of a sequence of extreme rainfall that persisted for about 5 days along the Front Range of Colorado in the headwaters of the Arkansas and South Platte Rivers. This sequence of rain resulted in large peaks in many southward- and northward-flowing streams in the Arkansas River basin near Colorado Springs and in numerous northward-flowing tributaries of the South Platte River. It also produced devastating floods on the Arkansas River downstream of Pueblo and on the South Platte River in Denver. Chatfield Dam was completed later to control floods on the South Platte River. The flooding is described by Snipes and others (1974) and is included in a report by Rostvedt and others (1970).

The June flooding in Colorado was front-page news in most area papers for several days preceding and following June 18. The Denver Post and Rocky Mountain News ran articles. However, pictures or discussion were not found of the Jimmy Camp Creek flood in these newspapers. Historical photographs taken after the June 17, 1965, flood and during the 2003 review and described herein are provided in figures A76–A81.

Method of peak discharge determination: The peak discharge is based on a two-section slope-area measurement. As part of the 2003 review, the original computation was coded for the present USGS slope-area computation program (SAC). The SAC peak discharge of 123,800 ft^3/s confirms the original discharge.

Fall in the slope-area measurement reach is large (12.22 ft of fall over the 1,680-ft reach. [**Note**: The new SAC program computed the average fall to be 12.25 ft]) and is well defined. Notes on the original computer output show that the water-surface slope of 0.00729 agrees with the channel slope over a 2.6-mi reach (0.00728). Agreement between the two profiles generally is good near the cross sections, and the right-bank profile fall is fairly uniform through the reach. The left bank, however, has a large "step" or fall in the middle of the reach. That fall is not explained in the measurement summary but may result from the channel alignment; the main channel appears to be curving to the right, which would direct flow into the left bank in that area. It is possible, given the flow direction and general topography, for the flow along the left bank to have essentially been "perched" for some distance and thus not reflect the water surface of the main part of the flow. That, however, is only speculation.

The reach contracts sharply; cross-section area decreases from about 14,000 ft^2 at section 1 to just less than 9,000 ft^2 at section 2. The conveyance change is even more pronounced with the conveyance at section 2 equal to only one-half the conveyance at section 1. The channel width is nearly equal at the two sections at about 2,900 ft. The cross sections were properly subdivided on the basis of shape. Section 1 had five subsections, and section 2 had six subsections. Alpha ranged from about 1.13 at section 1 to 1.66 at section 2.

The high degree of contraction in the reach produced high velocities in cross section 2 (25 ft/s in the main channel). Froude numbers indicate lower regime flow in all subsections of section 1, and upper regime flow in all subsections at cross section 2 (downstream section). The main channel carried about 30 percent of the flow, and the respective Froude numbers were 0.63 and 1.21.

Because of the break in slope in the middle of the reach, as part of the 2003 review, Kenneth Wahl (USGS retired) computed slope-conveyance estimates using each of the two sections and the local slopes at the sections. This was done to determine the uncertainty of the two-section result. Those slopes were identical (0.005); however, the conveyance of section 1 was about double that of section 2. The slope-conveyance results were 87,000 ft^3/s at section 2 and 171,000 ft^3/s at section 1. The square root of the multiple of these values is 128,000 ft^3/s.

Possible sources of error: The most likely sources of error in the measurement are in (1) the roughness values, (2) the assumptions that the post-flood cross section represented the cross section at the time of the peak discharge and that this two-section reach is representative, and (3) the assumption that energy losses are properly accounted for with a change in flow regime between the sections. The latter two assumptions are particularly critical, given the large fall and irregular left-bank profile in the reach that only spans about one channel width. The roughness values are consistent with verification data for sand-bed streams. Condition of the streambed during the peak discharge is unknown, but most of the streams in the Fountain area are known to transport large quantities of sand; there possibly could have been significant scour at the peak discharge relative to the post-flood channel.

Recommendations of what could have been done differently: Every effort probably was made to obtain more than two sections (a long reach was surveyed, but profiles did not support more than two sections). However, when it became evident that only two sections could be used at this location, another reach should have been sought, either as an alternative to this reach or as a supplement. Two independent two-section results would have given some measure of the reliability of the result.

Reviews are not included with the measurement summary. Kenneth Wahl knew that measurements for the 1965 floods in Colorado were done in assembly-line fashion, and all were reviewed. Those reviews, and the names of the reviewers, should have become a permanent part of the indirect measurement. The record of those reviews likely will not be found.

Site visit and review: The site was visited June 4, 2003, by John Costa (USGS Office of Surface Water), Joseph Capesius (USGS Colorado Water Science Center), John England (Bureau of Reclamation), Mark Smith (USGS Central Region), and Kenneth Wahl (USGS retired).

The site and many reaches of Jimmy Camp Creek have changed a great deal since the 1965 flood. The 1965 photographs show a wide main channel with raw, eroded banks; top width was almost 300 ft at section 1 and more than 100 ft at section 2. Field data from 1965 show that the main channel is relatively straight through the reach, staying near the left side of the valley, and the flood plain is nearly devoid of trees and brush. In 2003, the main channel width averaged perhaps 40 ft, and the main channel meandered over perhaps a 1,000 ft of width as it passed through the reach. In addition, there were a considerable number of what appeared to be mature cottonwood trees along the channel and in the flood plain.

Recommendation: The original peak discharge of 124,000 ft^3/s should be accepted as published, but the quality rating should be downgraded to poor.

A great deal of effort was expended in 1965 to obtain a longer reach, but the two-section result was the best that could be obtained at this site. Although the two-section result contains a high degree of uncertainty, there is no evidence of errors either in procedure or interpretation, and there was no new evidence available in 2003.

Figure A76. View looking downstream from about 200 feet above cross section 2, Jimmy Camp Creek at Fountain, Colorado, July 17, 1965. (Man is holding rod at high-water mark at cross section 2.)

Figure A77. View looking downstream from about 200 feet above cross section 1, Jimmy Camp Creek at Fountain, Colorado, July 17, 1965. (Man is holding rod at cross section 1.)

Figure A78. View looking upstream, Jimmy Camp Creek at Fountain, Colorado, July 17, 1965. (Man is holding rod at high-water mark at cross section 1, 250 feet right of left end of cross section.)

Figure A79. View looking downstream near cross section 2, Jimmy Camp Creek at Fountain, Colorado), June 4, 2003.

Figure A80. View looking upstream of main channel flood plain, Jimmy Camp Creek at Fountain, Colorado, June 4, 2003.

Figure A81. View looking downstream toward flood plain upstream of slope-area reach, Jimmy Camp Creek at Fountain, Colorado, June 4, 2003.

06759000 Bijou Creek near Wiggins, Colorado

(Discontinued gaging station, USGS Colorado Water Science Center)

Review of peak discharge for the flood of June 18, 1965

Location: This flood was located about 6.3 mi east of Wiggins, Colo., along the Interstate Highway 76 at 40.2547 N and 103.9667 W.

Published peak discharge: The published peak discharge was 466,000 ft^3/s, on June 18, 1965. The measurement was rated poor.

Drainage area: The drainage area of the original site for the streamflow-gaging station is 1,314 mi^2, which is 5.6 mi upstream of the location of the indirect discharge measurement for the June 18, 1965, flood. Drainage area at the indirect discharge measurement site is 1,500 mi^2.

Data for storm causing flood: The flood of 1965 was the result of a sequence of extreme rainfall that persisted for about 5 days along the Front Range of Colorado in the headwaters of South Platte River. Another large flood (stage of 15.9 ft) occurred at this site on June 15 according to the measurement summary. This sequence of rain resulted in large peak discharges in most of the northward-flowing tributaries of the South Platte River as well as producing devastating floods on the South Platte River upstream of Denver to the Colorado-Nebraska State line. Chatfield Dam was completed later to control floods on the South Platte River, primarily Plum Creek. The flooding is described by Matthai (1969) and is included in a report by Rostvedt and others (1970). Sediment deposits resulting from the flood were described by McKee and others (1967).

The June flooding in Colorado was front-page news in most area papers for several days preceding and following June 18, 1965. The Denver Post and Rocky Mountain News ran articles. Aerial photographs on the front page of the June 19 Fort Morgan Times and the Denver Post show the flooding on Bijou Creek at the former gaging station. The photographs provide graphic testimony about the size of this flood and the amount of embankment overtopped. Those photographs need to be seen to appreciate the scale of flooding. Historical photographs taken after the June 18, 1965, flood and during the 2003 review and described herein are provided in figures A82–A94.

A daily-discharge gaging station was operated at U.S. Highways 6 and 34 bridge (now I-76) just downstream of what is now the Burlington-Northern-Santa Fe (BNSF) railroad bridge from April 1, 1950, to September 30, 1956. The stream at the former gaging station is ephemeral, flowing only in response to thunderstorm activity and then only for a few days in most years. During the more than 6 years of record, flow never was recorded during October–April, and there were no periods when flow was recorded for more than 7 consecutive days. During the period of gaging, however, several large peaks were recorded as shown below:

Date	Discharge (ft^3/s)	Gage height (ft)
July 31, 1950	767	4.89
August 3, 1951	50,100	10.22
August 22, 1952	7,840	6.12
July 30, 1953	1,080	3.82
July 30, 1954	5,700	5.52
August 28, 1955	2,450	4.48
July 31, 1956	19,000	7.80

Method of peak discharge determination: The peak discharge is based on a three-section slope-area measurements made about 5.6 mi downstream of the site of the discontinued gaging station. Spread between the two subreach discharges computed for the measurement reach is only 16 percent. The channel width averages about 3,800 ft, and cross-section area averages about 30,000 ft^2 through the measurement reach. Fall in the reach is substantial (13.04 ft of fall over the 3,845-ft reach), but it is well defined by high-water marks. Agreement between the two profiles generally is good except on the right bank just upstream of section 2. Most right-bank fall in subreach 1–2 occurs in a single large fall just upstream of section 2. This fall probably relates to run-up as the main channel moves from the middle of the channel at section 1 to the right side of the channel at sections 2 and 3. However, the total fall in the reach could not be changed a great deal by any reasonable reinterpretation of the profiles. Matthai (1969) noted that the water-surface slope in the reach (0.0034) was comparable to the slope over a 2.3-mi reach of the channel from the Weldona Quadrangle (0.0033).

The cross sections were properly subdivided based on shape with each section broken into four subsections. Alpha was approximately 1.4 at all sections. The reach expands slightly from section 1 to 2 but contracts from section 2 to 3. However, the expansion is not a significant factor as there is only a 7-percent spread between computations for 0 and 100-percent energy recovery in the expanding reach. Velocities in the main channel are high, ranging from 21 ft/s at downstream section 3 to 26 ft/s at upstream section 1. Main channel Froude numbers of 1.34, 1.10, and 1.12 indicate upper regime and supercritical flow in all sections. The main channel carried about 40–45 percent of the flow.

As part of the 2003 review, the original computation was coded for the current USGS slope-area computation program (SAC). The SAC peak discharge (464,000 ft^3/s) confirms the original discharge.

According to the measurement summary, the June 18 peak discharge at this site may have been amplified by a release of water that ponded upstream of the BNSF railroad. The railroad embankment to the right of the railroad bridge failed. The measurement summary speculates that the failure could have been rapid and notes an earlier failure in 1935 that is discussed by Follansbee and Sawyer (1948, p. 71). However, newspaper accounts of the 1965 peak discharge talk about a large crest passing the community of Hoyt (about 20 mi upstream) in the early morning hours. The size of peaks from upstream tributaries, the amount of railroad embankment that was subjected to overflow (about 4,000 ft) relative to the fairly modest amount of embankment failure (hundreds of feet) suggest that the failure probably contributed little to the actual peak discharge. Aerial photographs on the front page of the June 19 Fort Morgan Times and the Denver Post need to be seen to appreciate the scale of flooding and the amount of embankment overtopped. The photographs show only a very few trees in the reach downstream of the railroad/Interstate crossing; in 2003, nearly mature cottonwood trees were scattered in this reach.

Possible sources of error: The most likely sources of error in the measurement are (1) the roughness values, (2) the assumption that the post-flood cross section represented the cross section at the time of the peak discharge, and (3) the possible effect of the railroad embankment failure. The roughness values were based on bed-material samples (median size 0.44 mm) and are consistent with verification data for high-gradient, sand-bed streams. Condition of the streambed during the peak is unknown, but Bijou Creek is known to transport large quantities of sand; significant scour could have occurred during the peak discharge relative to the post-flood channel. The effect on the peak of the embankment failure is believed to be small for the reasons noted in the previous paragraph.

Recommendations of what could have been done differently: The summary for this important indirect measurement has never been typed. Reviews are not included with the measurement summary. The writer knows that measurement of the 1965 floods in Colorado was done in assembly-line fashion, and all were reviewed. Those reviews, and the names of the reviewers, should have become a permanent part of the indirect measurement. The record of those reviews likely will not be found. A file of the newspaper coverage complete with photographs should be a part of the permanent record.

One thing that was done correctly was to document many peak discharges from the flood instead of just a few. The evidence of many extreme peak discharges is compelling corroboration for the individual peak discharges.

Site visit and review: The site was visited June 3, 2003, by John Costa (USGS Office of Surface Water), Joseph Capesius (USGS Colorado Water Science Center), John England (Bureau of Reclamation), Mark Smith (USGS Central Region), and Kenneth Wahl (USGS retired). The visit included stops at the BNSF railroad and Interstate Highway 76 crossing (the former gaging station) and the indirect measurement site about 6 mi downstream.

The reach used for the indirect measurement has changed little since 1965. Flood debris is still evident at places in the measurement reach, which has scattered cottonwood trees on the flood plain. The 1965 photographs show a sand bed in the main channel and the overflow sections. The sand is still present but has been overgrown with grass and small shrubs. Land-use changes upstream have produced a very slight base flow in the reach; as a result, the main channel now has pooled water, cattails, and reeds.

Recommendation: The original peak discharge of 466,000 ft^3/s should be accepted as published.

Photographic and geomorphic evidence leaves no doubt that this was a water flood. The indirect measurement was done correctly, and there is no evidence of error either in procedure or in interpretation.

For some reason, the measurement summary that is part of the indirect measurement has never been typed. The summary for this very unusual flood should be typed and properly archived. The aerial photographs from the Fort Morgan Times and the Denver Post should become a part of the permanent record of this indirect measurement.

Figure A82. View looking upstream from cross section 3, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A83. View looking downstream from 50 feet upstream of cross section 2, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A84. View looking downstream from 100 feet upstream of cross section 2, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A85. View looking downstream from 100 feet upstream of cross section 3, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A86. View looking downstream from 150 feet upstream of cross section 2, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A87. View looking downstream from 200 feet upstream of cross section 1, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A88. View looking downstream from 200 feet upstream of cross section 1 (different location than shown in figure A87), Bijou Creek near Wiggins, Colorado, June 1965.

Figure A89. View looking downstream from 200 feet upstream of cross section 2, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A90. View looking upstream from 200 feet downstream of cross section 1, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A91. View looking downstream from 150 feet upstream of cross section 1, Bijou Creek near Wiggins, Colorado, June 1965.

Figure A92. View looking north toward railroad wash-out, Bijou Creek near Wiggins, Colorado, June 3, 2003.

Figure A93. June 1965 flood debris in slope-area reach, Bijou Creek near Wiggins, Colorado, June 3, 2003.

Figure A94. Flood plain of Bijou Creek near Wiggins, Colorado, in slope-area reach, June 3, 2003.

East Bijou Creek at Deertrail, Colorado

(Miscellaneous ungaged site in the South Platte River basin, USGS Colorado Water Science Center)

Review of peak discharge for the flood of June 17, 1965

Location: This flood was located just downstream of the town of Deertrail, Colo., at 39.6132 N and 104.0504 W.

Published peak discharge: The peak discharge from the indirect measurement is 274,000 ft^3/s, June 17, 1965, and the measurement was rated fair.

Drainage area: 302 mi^2. The map scale used to define the drainage area is unknown.

Data for storm causing flood: The flood of 1965 was the result of a sequence of extreme rainfall that persisted for about 5 days along the Front Range of Colorado in the headwaters of the South Platte River. This sequence of rain resulted in large peak discharges in most of the northward-flowing tributaries of the South Platte River as well as producing devastating floods on the South Platte River in Denver and downstream. Chatfield Dam was completed later to control floods on the South Platte River. The flooding is described by Matthai (1969) and is included in a report by Rostvedt and others (1970). Sediment deposits resulting from the flood were described by McKee and others (1967). This latter article included a site on East Bijou Creek near Highway 36, about 10 mi downstream of Deertrail.

The June flooding in Colorado was front-page news in most area papers for several days preceding and following June 18. The Denver Post and Rocky Mountain News ran articles. The June 18 edition of the Denver Post has photographs of the destruction of Interstate Highway 70 and railroad bridges at Deertrail (East Bijou Creek) and at Byers (West Bijou Creek). East Bijou Creek is ephemeral, flowing only in response to thunderstorm activity and then for only a few days in most years. Historical photographs taken after the June 17, 1965, flood and during the 2003 review and described herein are provided in figures A95–A99.

Method of peak discharge determination: The peak discharge is based on a four-section slope-area measurement. The channel top width ranges from about 3,300 to about 4,000 ft, and the cross-sectional area averages about 30,000 ft^2 through the measurement reach. Maximum spread between subreach discharges is about 33 percent, but the spread between three-section results is only 6 percent.

Fall in the reach is substantial (11.95 ft of fall over the 3,450-ft reach), but it is fairly well defined. This slope (0.0034) is consistent with the slope downstream on Bijou Creek at Wiggins, Colo. Matthai (1969) noted that the water-surface slope in the reach at Wiggins (0.0034) was comparable to the slope over a 2.3-mi reach of the channel from the Weldona Quadrangle (0.0033). Agreement between the left- and right-bank profiles generally is good except on the left bank just upstream of section 3 where there is an apparent fall of about 5 ft. However, the total fall in the reach could not be changed a great deal by any reasonable reinterpretation of the profiles.

The summary notes that the cross sections were subdivided "… primarily on the basis of ground cover …"; however, those subdivisions match what would have been done if based on shape. Each section is broken into four subsections. Alpha ranged from 1.89 at section 1 to 1.55 at section 4.

As part of the 2003 review, the original computation was coded for the current USGS slope-area computation (SAC) program. The SAC peak discharge of 274,300 ft^3/s confirms the original peak discharge. The reach expands slightly from sections 1 to 3 but contracts from sections 3 to 4. The spread between 0 and 100 percent energy recovery is 12 and 21 percent, respectively, in subreaches 1–2 and 2–3. However, the SAC analysis shows only an 8-percent spread between computations for 0- and 100-percent energy recovery in the multisection result. Velocities in the main channel are high, ranging from about 17 ft/s at section 3 to about 22 ft/s at section 1. Main channel Froude numbers of 1.03, 0.99, 0.85, and 1.03 indicate that flow probably is near critical flow at all sections. The main channel subsection carries about 45 percent of the total flow.

Possible sources of error: The most likely sources of error in the measurement were in the roughness values and the assumption that the post-flood cross section represented the cross section at the time of the peak. The roughness values are consistent with verification data for steep, sand-bed streams, but the summary notes some question about the roughness of the parts of sections 1–3 located in the town of Deertrail, Colo. Condition of the streambed during the peak discharge is not known, but large quantities of sand were transported; significant scour could have occurred during the peak discharge. Because of the extreme width and relatively shallow depths and because the railroad embankment traverses the length of the reach, there is some question about the applicability of the assumption of one-dimensional flow.

Recommendations of what could have been done differently: Reviews are not included with the measurement summary. Kenneth Wahl (USGS retired) stated that measurements of the 1965 floods in Colorado were done in assembly-line fashion, and all were reviewed. Those reviews, and the names of the reviewers, should have become a permanent part of the indirect measurement. The record of those reviews likely will not be found. A file of the newspaper coverage complete with photographs should be a part of the permanent record.

Site visit and review: The site was visited June 3, 2003, by John Costa (USGS Office of Surface Water), Joseph Capesius (USGS Colorado Water Science Center), John England (Bureau of Reclamation), Mark Smith (USGS Central Region), and Kenneth Wahl (USGS retired). The visit included a drive-by of the railroad and Interstate Highway 70 bridge crossing several miles downstream (featured in the June 18, 1965, Denver Post).

The reach used for the indirect measurement has changed little since 1965. Flood debris is still evident at places in the measurement reach, which has scattered cottonwood trees on the flood plain. However, many of those trees are clearly less than 40 years old. The 1965 photographs show a sand bed in the main channel and the overflow sections. The sand is still present but has been overgrown with grass and small shrubs. There is now a small base flow in the reach; as a result, the main channel now has pooled water and some exposed gravel.

The railroad embankment actually enhanced the possibility of one-dimensional flow. The embankment parallels the plan-view baseline and effectively served as a submerged levee, directing the flow in the downstream direction.

Recommendation: The original peak discharge of 274,000 ft^3/s should be accepted as published.

Photographic and geomorphic evidence leaves no doubt that this was a water flood. The indirect measurement was done correctly, and there is no evidence of error either in procedure or in interpretation.

Figure A95. View of flood plain looking toward channel in slope-area reach, East Bijou Creek at Deertrail, Colorado, June 1965.

Figure A96. View looking across flood plain toward left bank high-water mark (woody debris), East Bijou Creek at Deertrail, Colorado, June 3, 2003.

Figure A97. View of flood plain looking downstream with main channel in tree line, East Bijou Creek at Deertrail, Colorado, June 3, 2003.

Figure A98. View from left valley side looking across flood plain and main channel toward slope-area reach, East Bijou Creek at Deertrail, Colorado, June 3, 2003. (Flow is from right to left.)

Figure A99. Walking on flood-plain surface toward main channel in slope-area reach, East Bijou Creek at Deertrail, Colorado, June 3, 2003. (Flow was 5-6 feet deep at this point during the 1965 flood.)

Lahontan Reservoir Tributary No. 3 near Silver, Nevada

(Miscellaneous ungaged site near Silver Springs, Nevada;
USGS Nevada Water Science Center)

Review of peak discharge for flood of July 20, 1971

Location: This flood was located about 3 mi south of Silver Springs, Nev., at 39.3616 N and 119.2752 W.

Published peak discharge: The peak discharge for this site is 1,840 ft^3/s and was rated fair. The rating should be downgraded to estimate. A discharge of 1,680 ft^3/s (the original hand-calculated value) is published in Moosburner (1978).

Drainage area: The drainage area originally was estimated at 0.22 mi^2 by planimeter from the Churchill Butte quadrangle map, scale 1:24,000.

Data for storm causing flood: The storm is described by Patrick Glancy (USGS retired), as a high-intensity thunderstorm with more intense inner cells. Data on precipitation were not gathered as part of this review. A rain gage at nearby Lahontan Dam, 12 mi east of the slope-area measurement site, collected a little more than 1 in. of rain on July 19 and 0.37 in. on July 20, which probably do not reflect the rainfall amounts or storm intensities in the area. Lahotan Reservoir is on the flat valley floor at an elevation of less than 1,300 ft above sea level. Precipitation in the headwaters, at elevations greater than 1,600 ft, probably was greater than at Lahontan Dam. Historical photographs taken after the July 20, 1971, flood and during the 2003 review and described herein are provided in figures A100-A105.

Method of peak discharge determination: The peak discharge for this site was determined by a two-section slope-area measurement. Data for this calculation was collected on August 18, 1971.

The reach was selected because it is straight and is one of the few reaches where all flow was confined to one channel. The high-water profiles are uniform and well defined by an appropriate number of high-water marks although they were rated fair to poor in quality. The profiles are parallel to each other and to the channel slope. The cross sections were correctly located to minimize the effect of channel bends upstream and downstream of the reach. The slope-area measurement paperwork describes a channel bar at and upstream of section 1. Section 1 was located near the toe of this bar and was subdivided on the basis of shape. Section 2 did not require subdivision. The cross-sectional end elevations were picked from profile interpretation between high-water marks, but the marks are close enough together that getting marks on the cross section would not have increased the accuracy of the calculation. The 58-percent expansion decreases velocity from 30 to 15 ft/s, and Froude numbers ranged from 3.4 to 2.6 from section 1 to section 2, indicating supercritical flow.

The streambed is erodible and underlain by fractured bedrock that is exposed on the left bank at both cross sections. There was potential for a substantial amount of sediment to be transported through the reach but downcutting probably was limited by the bedrock. Channel sediment is mostly sand and gravel as much as about 1.5 in. diameter. Manning's "*n*" values were 0.035 and 0.037, respectively, for sections 1 and 2. The subsection at section 1 was assigned an "*n*" value of 0.044. Flow depths were in the range 3 to 4 ft, and the slope has a high gradient (a fall of 7.38 ft in 95 ft or a slope of 0.075 ft/ft).

Two errors were found in the original hand calculation. The total cross-sectional area was used as the area of the subsection for subdivided section 1. An extra digit was read from the calculator screen when computing conveyance for the same section. Win Hjalmarson (USGS Arizona Water Science Center) discovered the area error during his review of this indirect measurement in 1988. His recalculation, using a prior version of the USGS slope-area computation (SAC) program, yielded a discharge of 1,830 ft^3/s. Calculation using the current version of SAC produced the same discharge. The decrease in area did not explain the increase in discharge. A recheck of the hand calculation identified the conveyance error. The hand calculation agreed with the SAC results after the area and conveyance errors were corrected.

Possible sources of error: The most probable source of error is in selection of roughness coefficients for steep, movable-bed streams. The values used seem consistent with verified coefficients for streams that are less steep. The revised Froude numbers are high, ranging from 3.38 to 2.21, which appear unrealistically high. The drainage area is a possible source of error. Previous reviewers have questioned the location of the reach. Unit discharge is sensitive to basin size in drainages this small. The USGS Nevada Water Science Center used the GPS site-location data collected by the field-review team to remeasure the drainage area. The result was not significantly different from the original value. The excessive expansion and high Froude numbers also are a concern. Because of the "bar" at section 1, the conveyance did not vary uniformly between sections. The basin is highly erodible, so hyperconcentrated flows could have occurred.

Prior reviews suggest reducing the discharge for this flood to 700 ft^3/s on the basis of the assumption that movable-bed streams tend to adjust to critical flow. This change also would incorporate using the Jarrett (1984) equation to compute an effective “n-value” of about 0.14. This equation has not been verified for streams with movable beds or for slopes this steep. The opinion of the field-review team is that the relatively small part of the bed that could become mobile and the probable short duration of high flow make critical flow a questionable argument.

Recommendations for what could have been done differently: There is little that could have been done to improve this measurement. Some digging might have shed light on potential depths of scour. A more exact field location description would have been valuable. A more thorough review would have caught the two errors that were identified in this review. The sections are about 50 ft wide, the reach length between sections is 95 ft, so a third section could have been added to help assess the reliability of the peak discharge. Additionally, the extreme unit discharge warranted a return visit to try to find sites for indirect measurements in tributary or adjacent drainages to help validate the Lahontan Reservoir tributary no. 3 flood discharge.

Site visit and review: The site was visited on July 31, 2003, by John Costa (USGS Office of Surface Water), Patrick Glancy (USGS retired), Kerry Garcia (USGS Nevada Water Science Center), and Gary Gallino (USGS retired). The site was approximately located a week earlier by Kerry Garcia and Bob Burrows (USGS). This effort saved valuable field time. The original hubs and cross-section stakes were found, and a GPS reading of latitude and longitude were taken to positively locate the reach. These readings were used with the most recent topographic map to check the drainage area. The reach appears to have changed little when compared to photographs (stereo slides) taken shortly after the flood. Extensive side-hill erosion scars are evident in the upstream part of the basin and are visible in slides taken by Patrick Glancy documenting the original flood. The basin appears to have a history of erosion and high unit discharge. There is no evidence that this flood was a debris flow.

Recommendations: The original peak discharge of 1,680 ft^3/s should be updated to 1,840 ft^3/s and the rating should be downgraded to “estimate” because of the unrealistically high Froude numbers and excessive expansion. This value agrees with results from the corrected hand calculation and the prior SAC analysis.

Figure A100. View looking downstream of slope-area reach, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 1971.

Figure A101. View looking upstream at cross section 2, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 1971.

Figure A102. View looking upstream of slope-area reach, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 1971.

Figure A103. View looking downstream of slope-area reach, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 31, 2003.

Figure A104. View looking upstream toward cross section 2, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 31, 2003.

Figure A105. View looking upstream of slope-area reach, Lahontan Reservoir tributary no. 3 near Silver, Nevada, July 31, 2003.

10335080 Humboldt River Tributary near Rye Patch, Nevada

(Miscellaneous ungaged site in the Humboldt River basin, USGS Nevada Water Science Center)

Review of peak discharge for flood of May 31, 1973

Location: This flood was located about 20 mi northeast of Lovelock, Nev., at 40.4196 N and 118.2573 W.

Published peak discharge: The published peak discharge for this flood is 8,870 ft^3/s and was rated fair. The peak discharge should be downgraded to estimate. The discharge published in Moosburner (1978), written in cooperation with the Nevada State Highway Department, is 8,940 ft^3/s and is thought to be a typographical error. The original hand-calculated discharge was 8,960 ft^3/s, but minor errors discovered in review reduced the discharge to 8,870 ft^3/s.

Drainage area: The drainage area, as originally planimetered from the 1:24,500-scale Oreana and Unionville quadrangle maps, is 0.85 mi^2.

Data for storm causing flood: The storm was reported by Patrick Glancy (USGS retired) to be an area-wide storm with intense inner cells that produced high runoff from several basins draining the west-facing slopes of the Humbolt Range. This is a sparsely populated area, and precipitation data may be available only at Rye Patch Reservoir, about 25 mi north of the slope-area measurement site. Precipitation data were not available for this review. Historical photographs taken after the May 31, 1973, flood and during 2003 review and described herein are provided in figures A106–A118.

Method of peak discharge determination: A four-section slope-area measurement was run on June 6, 1973, at a site selected by Howard Matthai and Lynn Harmsen (USGS). The reach is straight, high-gradient, and all flow was confined to one channel. The high-water profiles are well defined by an appropriate number of high-water marks, but wash and debris lines were fair to poor. Sheet flow over the length of the canyon, described as "side hill wash," reduced the quality of many high-water marks. Fall through the reach is almost 30 ft in the 400 ft length from sections 1 to 4. The bed consists of sand, gravel, and cobbles with scattered large boulders. Bedrock is exposed at section 3 and may underlay the entire reach with only a thin veneer of erodible sediment. Flow is reported to have extended about 0.25 mi across the flat outflow plain causing closure of Interstate Highway 80. The outflow plain is littered with huge boulders (Volkswagen size) that are evidence of past major floods.

The cross sections are nearly trapezoidal, fairly uniform in size and shape, and correctly located. There is no evidence of significant downcutting in the reach. The exposed bedrock indicates that the channel is fairly stable. Sections 1–3 were not subdivided. Section 4 was subdivided on the basis of shape because of a small shallow area on the left bank. A Manning's roughness coefficient of 0.032 was used for the entire reach with the exception of the small shallow subarea at section 4. A value of 0.065 was used for this subsection. These values seem reasonable based on depth of flow, bed-material size, and the lack of any significant bank irregularities. Main channel velocities were calculated to be just over 30 ft/s and Froude numbers ranged from 2.2 to 2.8, which appear unreasonably high.

Slides taken by Patrick Glancy (USGS retired) documenting this flood show extensive erosion scars in the upstream part of the basin. The flood transported a large amount of sediment through the reach after this flood as evidenced by extensive fresh deposits of sediment on the shallower gradient receiving river downstream of the mouth of the canyon.

A slope-area measurement for runoff from the same storm was made about 1 mi south at Rocky Canyon near Oreana, Nev. The reach at that site was poor, and the runoff profiles were erratic with evidence of several feet of extremely high elevation. The discharge was 14,400 ft^3/s from the 4.05 mi^2 drainage basin with a unit runoff of 3,550 $(ft^3/s)/mi^2$. A slope-area analysis in the same basin, 1.1 mi upstream of the mouth of Rocky Canyon, yielded a peak discharge of 683 ft^3/s from a 3.02-mi^2 drainage basin. The unit discharge at this site was 228 $(ft^3/s)/mi^2$. These two indirect measurements demonstrate the wide range in unit runoff in a small area as a result of storms with high-intensity cells.

Possible sources of error: The most probable source of error is selection of roughness coefficients for steep-gradient streams with movable beds. The roughness coefficients used are consistent with verified values for streams with similar bed material and gentler slopes. The drainage area is small so any error in location or drainage boundary will have a significant effect on unit discharge. The 30-ft/s velocities and high Froude numbers (2.2–2.8) are a concern, but with steep slopes and shallow depths (about 6 ft), they may be realistic. These values are comparable to those computed for other steep-gradient streams in arid and semiarid regions. The stream moved a lot of sediment, but there is no evidence that this was a debris flow, although there may have been hyperconcentrated flow.

Recommendations of what could have been done differently: This slope-area measurement is correctly done, and the reach is about as good as any in this environment. A second indirect measurement could have been made farther up the canyon or in one of the tributary canyons to verify the high unit discharge, particularly when the slope area in Rocky Canyon produced a much smaller unit runoff. Precipitation data may have been helpful in adding validity to the runoff, but there may not have been much data available.

Site visit and review: The site was visited on July 30, 2003, by John Costa (USGS Office of Surface Water), Patrick Glancy (USGS retired), Kerry Garcia (USGS Nevada Water Science Center), and Gary Gallino (USGS retired). The site was approximately located by Kerry Garcia and Bob Burrows (Nevada Water Science Center) about a week earlier and saved valuable field time. The original cross-section stakes and hubs were found, and GPS readings of latitude and longitude were taken to positively locate the reach. These readings will be used with the most recent topographic map to check the drainage area. The reach appears to have changed very little when compared to slides taken shortly after the flood. Extensive side-hill erosion scars are evident in the upstream part of the basin and are visible in stereo slides taken by Patrick Glancy documenting the flood.

Recommendations: The original peak discharge of 8,870 ft^3/s should be accepted as published and the rating should be downgraded to “estimate” because of the unusually high Froude numbers.

The published value of 8,940 ft^3/s (Moosburner, 1978) is a typographical error and should be corrected.

Figure A106. View looking downstream of range marker 1 (painted white spot on rock), Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A107. View looking downstream of hill near left end of cross section 3, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A108. View looking downstream through slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A109. View looking upstream at right bank through cross section 3, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A110. View looking upstream of hilltop near left end of cross section 3, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A111. View looking upstream through cross section1 overbank flow area, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A112. View looking upstream through slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, June 1973.

Figure A113. View looking downstream through slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003.

Figure A114. View looking downstream through slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003. Rock with painted registered mark shown on left.

Figure A115. Original registered mark (white paint on rock) found 30 years later, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003.

Figure A116. View looking downstream of top of slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003. People standing at cross sections.

Figure A117. View looking upstream through slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003.

Figure A118. View of left bank near top of slope-area reach, Humboldt River Tributary near Rye Patch, Nevada, July 30, 2003.

Eldorado Canyon at Nelson Landing, Nevada

(Miscellaneous ungaged site, USGS Nevada Water Science Center)

Review of peak discharge for the flood of September 14, 1974

Location: This flood was located about 19 mi southeast of Boulder City, Nev., at 35.7066 N and 114.7148 W.

Published peak discharge: The computed peak discharge is 75,800 ft^3/s and was rated poor. This was published as 76,000 ft^3/s in Glancy and Harmsen (1975). The rating should be downgraded to estimate.

Drainage area: Drainage area at the indirect measurement site is listed as 22.8 mi^2. Glancy and Harmsen (1975) list the drainage area of Eldorado Canyon as 22.9 mi^2 at the mouth. However, most runoff was generated in the central and downstream part of the basin.

Data for storm causing flood: The flood was the result of an intense convective thunderstorm that moved slowly down the drainage basin. The storm and subsequent flooding are summarized in Glancy and Harmsen (1975), National Weather Service (1974), and Cleveland (1975).

The National Weather Service report (1974, p. 3) states that the flood

> "…was caused by a classical convective runoff-producing event. Area coverage was small — less than 50 square miles. Duration of rainfall was short — generally less than one hour. Intensities were very high — at least three inches per hour, and as high as 7 inches per hour for ½ hour."

The down-basin movement of the storm intensified the flooding. According to local observers referenced in the previously cited reports, the storm lasted less than 1.5 hours, with maximum intensity spanning less than 0.5 hour. Storm totals of about 1.9 in. were reported at Nelson Landing. The National Weather Service (1974, p. 3) report also notes

> "The entire 23 square mile basin appears to have received over one inch of rainfall, with the storm center receiving at least 3.50 inches."

Given the magnitude and unit discharge of this flood, it is likely that much more rain fell in parts of the basin.

At least nine people were killed by the flood as it passed through the marina area where Eldorado Canyon enters Lake Mojave. As a result, there was extensive newspaper coverage of the flood and the recovery efforts. The Nevada State Journal, Las Vegas Review-Journal, and the Las Vegas Sun all carried articles about the flood in their September 16–18, 1974, issues. Articles about the recovery of flood victims noted that all bodies recovered were nude, their clothing having been completely stripped by the force of the flow.

The September 17 issue of the Las Vegas Review-Journal carried a photograph (fig. A119) of the flood at the resort with the caption,

> "The photograph was taken by a witness who ran to high ground and pointed his camera down the canyon."

The same photograph is figure 7*A* of Glancy and Harmsen (1975) and has the caption

> "On September 14, 1974, probably during the recession of flooding (photograph by Kenneth E. Beales, Las Vegas, Nevada)."

The photograph shows a water flood. However, the newspaper caption suggesting that the photograph was taken before the peak is in error. The restaurant is missing, giving graphic testimony that the photograph was taken after the destruction of the restaurant.

All eyewitness accounts mention that the flood arrived as a "wall of water" laden with debris, but the details of the height of this "wall of water" vary. What is not clear from those accounts is whether the "wall of water" was in the form of a wave in which the following flow was at a lower level or if it was just the snout of the floodflow that followed. It is very rare for any eyewitness account (or subsequent newspaper coverage) of a flash flood to be described as other than a "wall of water." Glancy and Harmsen (1975, p. 9) note,

> "The flow rate and velocity of the damaging initial flood surge at Eldorado Canyon cannot be determined because later flow apparently erased high water lines of the initial surge."

If true, this indicates that the flood stage of subsequent flows surpassed the stage of the initial surge.

Cleveland (1975, p. 54) notes,

> "Considerable speculation exists regarding the crest of the flood at the landing. Some observers reported unrealistic heights of the water surface… . Some mobile homes parked only a few feet above the floor of the canyon were not reached by the flood waters. Yet elsewhere downstream, floating debris was carried up to about 30 feet, perhaps by surges of water meeting natural obstacles along the canyon walls."

Figure A119. Flood in Eldorado Canyon at Nelson Landing, Nevada, September 14, 1974 (from Glancy and Harmsen, 1975, fig. 7*A*). Photograph taken by Kenneth E. Beales, Las Vegas, Nevada.

The canyon would be expected to experience higher velocities than the slope-area reach because flow is more greatly contracted in the canyon. Average velocity at the downstream cross section of the slope-area reach was 39 ft/s. That velocity converts to a static head of 23.6 ft, which would contribute to a similar depth of run-up on channel-bank protuberances, obstructions, and around channel bends. Historical photographs taken after the September 14, 1974, flood and during the 2003 review and described herein are provided in figures A120–144.

Method of peak discharge determination: The discharge is based on a three-section slope-area measurement surveyed September 17, 1974. The survey was made at a site upstream of the road where it switchbacks into Nelson Landing. A 780-ft reach was surveyed, and the three cross sections covered a 556-ft reach and fall was extreme; total fall was 30.32 ft. However, the reach was sharply contracting, and about one-half the fall was attributed to change in velocity head.

Velocities and Froude numbers were large. Mean velocity ranged from 25 ft/s at the upstream section (number 1) to 39 ft/s at the downstream section (number 3); Froude numbers were 1.56, 2.22, and 2.58 at sections 1–3, respectively.

As part of this review, the original results were analyzed using the current slope-area computation (SAC) program. The results confirmed the peak discharge of 76,000 ft^3/s computed in 1974.

The measurement summary notes that five slope-conveyance studies were done upstream to help define sources and magnitude of the flooding. Those five sites are described as one on Eagle Wash, two on Eldorado Canyon, one on Tachatticup Wash, and one on Morning Star Wash. Glancy and Harmsen (1975; table 2) give results for three sites including Eagle Wash, Tachatticup Wash, and Eldorado Wash upstream of the confluence with Eagle and Tachatticup Washes. Those results, assuming coincidence of the peak discharges and inflow from the unmeasured areas, are noted as giving credence to the value of about 76,000 ft^3/s. The actual computations for the slope-conveyance measurements could not be located during the 2003.

Because of the significance of the flood and the uncertainties in the computation, the measurement received extensive outside review. Howard Matthai (USGS) reviewed the measurement (and apparently the other slope-conveyance estimates as well) on October 8, 1974, and stated,

> "… the writer concludes that the only discharge figure that can be used is 75,800 cfs, but it may be 10 to 20 percent too high. The latter conclusion is based primarily on the extremely high velocities indicated for the discharge computed."

Howard Matthai had second thoughts after hearing the eye-witness accounts dealing with the "consistency" of the floodborne material and added an October 10, 1974, postscript to his initial review stating,

> "Under these conditions, I do not believe we can hide behind the idea that that we computed a discharge figure but do not claim it is all water. I am more convinced than ever that the discharge was not 75,800 cfs. If more evidence supports the 'semi-mud flow' condition, I would recommend that we report the peak discharge as indeterminate. If a discharge is needed, I suggest we use 20,000 cfs as a poor estimate."

Matthai then forwarded the measurement to USGS Headquarters where it was reviewed by Jack Davidian of the USGS Office of Surface Water. Davidian's review, dated October 25, 1974, recounted the uncertainties in the hydraulics of the flow and concluded,

> "Much of the above discussion is academic. All indications are that the flow was highly unsteady, and for such a condition we have no good means of measuring peak discharges. … It is definitely not recommended to give a discharge figure and qualify it with many reasons why it could be in error; it is far better to give no discharge figure, and explain the lack of it with those same reasons."

As a result of the uncertainties, Howard Matthai and Carl Nordin of the USGS research program (and one of the world's leading sediment transport experts) visited the site. That visit is documented in a hand-written note to the record by Lynn Harmsen dated November 20, 1974:

> "As a result of the review of the slope area determination by J. Davidian, Surface Water Branch, Washington, D. C., Mr. Matthai (W. R. Flood Specialist), Mr. Carl Nordin (Research Hydrologist, Denver), Mr. P. Glancy and Mr. L. Harmsen (the Carson City District Office) met in Las Vegas, Nev. on Nov. 18 to discuss the results and visit the sites in question. After careful inspection of the slope area site and the slope conveyance sites, the conclusion reached was that the numbers obtained were alright to use in the report as long as they were highly qualified. As to the probability of a gravel bar moving though the reach, there was no field evidence of this occurring."

Howard Matthai and Carl Nordin documented their reviews of the draft that would become Professional Paper 930 in memoranda dated November 25 and November 27, respectively.

In an April 28, 1983, memorandum to Patrick Glancy, Robert Jarrett of the USGS research program provided commentary on the indirect measurement. He echoed the concerns expressed earlier by Matthai and Davidian and computed flow at each cross section assuming that the flow was at critical depth. His results for total flow (sediment-water mixture) ranged from 47,700 ft^3/s at section 1 to 29,000 ft^3/s at section 3. He made the added assumption that this flow was 50-percent sediment by volume and gave an estimated water flood of 18,400 ft^3/s.

Kyle House, research geologist with the Nevada Bureau of Mines and Geology, summarized a reconnaissance study of this flood in a June 10, 2002, document presented to the Bureau of Reclamation. House estimated flow in an upper basin tributary (Huse Spring) using paleoflood-type discharge reconstruction and extrapolated that unit runoff to the entire basin to arrive at a flow of about 18 percent of the 76,000 ft^3/s. House notes, however, that the Huse Spring site is not representative of the basin as a whole and states,

> "…this unit runoff value is low with respect to what is likely to have characterized the lower part of the basin during the flood."

House further states,

> "…the estimate from Huse Spring … does not indicate that the estimate of 2,152 m^3/s from the canyon mouth is too large."

Possible sources of error: The possible sources of error have been documented in the several earlier reviews of this measurement. They include:

- Flow may have been unsteady, perhaps even in the form of translatory waves, rather than gradually varied as assumed by the slope-area procedure.
- A gravel bar may have been moving though the reach, affecting high-water mark placement.
- Unknown but obviously very high sediment concentrations.
- Possibility of a debris flow.
- Unaccounted for energy losses in the sharply contracting reach.
- Extreme velocities (25 to 39 ft/s) and suspect Froude numbers (ranging from 1.5 to 2.5).
- Unknown condition of the streambed at the peak discharge (scour/fill).

Recommendations of what could have been done differently: It is difficult to conceive what more could have been done. The indirect determinations were made within days of the flood while evidence was fresh. Slope-conveyance estimates were made at contributing reaches to corroborate the result. The credibility of the tributary results would have been enhanced if they had been slope-area measurements instead. However, the results would not have been likely to change appreciable, given the uniformity of the reaches. Because these tributary results are independent of one another, it is unlikely that all could be grossly overestimated unless *n*-values are much too small. The computations and the report documenting those results were reviewed extensively by some of the most experienced flood and sediment specialists in the country. Finally, USGS Professional Paper 930 that documented the results and the uncertainties in those results was published within about 6 months.

Harry H. Barnes, Jr., then Chief of the Surface Water Branch captured the situation nicely with his December 26, 1974, memorandum recommending approval of the Glancy and Harmsen report:

> "This is a good report and is more deserving than open-file status. I suggest it be published in the Professional Paper series. The flood in Eldorado Canyon is somewhat unique only because of the tragic consequences. From a hydrologic point of view a flood of this nature is probably not uncommon considering the West as a whole — yet the reader will be impressed by the complexity of the hydrologic and hydraulic processes that produced the event and the uncertainties of post flood analysis and documentation."

The methods used to document this flood would most likely be used if a similar flood happened today. Although some would call for use of a two-dimensional model instead of the one-dimensional model that comprises the slope-area method, unless there was definitive data (on a time scale of seconds) on the actual flood wave, a two-dimensional model would permit only generation of a larger number of alternative peak discharges for debate.

Site visit and review: A field visit was made August 28, 2003, by John Costa (USGS Office of Surface Water), and Kyle House, Gary Gallino, Patrick Glancy, Robert Burrows, Terry Kenney (USGS), and Kenneth Wahl (USGS retired). Primary focus of the visit was the slope-area reach and former site of Nelson's Landing, but the general reaches of the Tachatticup and Eagle Wash slope-conveyance sites were viewed from the basin-perimeter roadway.

The slope-area reach is remarkably unchanged from the conditions present in 1974. There appears to have been very little net change in either the streambed or the banks. This is consistent with the observation that steep channels like this may act somewhat like conveyors during large flows—moving large amounts of bed material with little accumulation on or erosion from the streambed. Those present at the field visit agreed that there was no evidence that the 1974 peak had been a debris flow at the indirect measurement site.

Recommendations: The original peak discharge of 76,000 ft^3/s should be accepted as published and the rating should be downgraded to "estimate" because of the extraordinary Froude numbers.

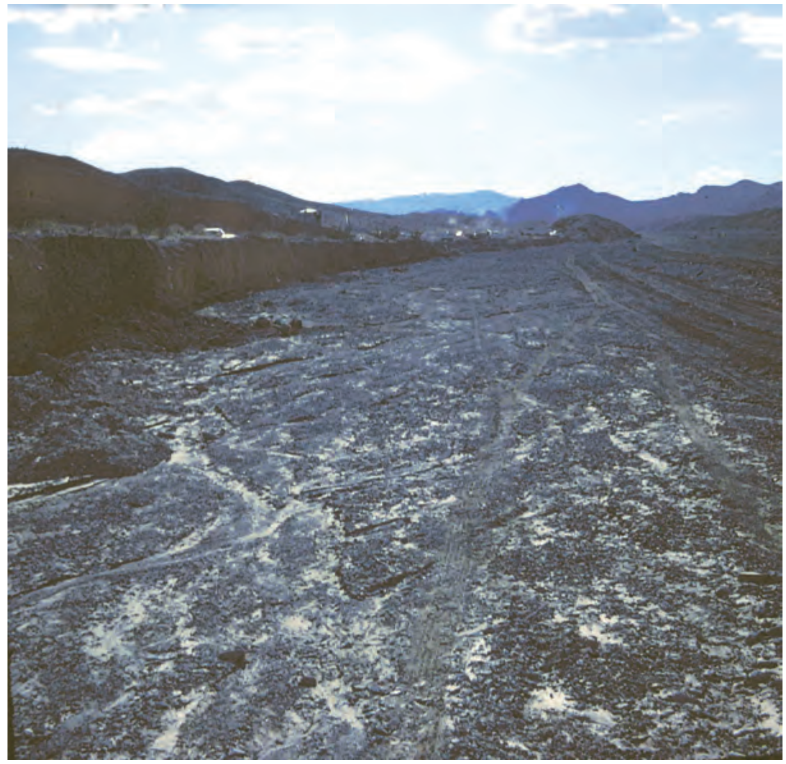

Figure A120. View looking upstream through slope-area reach, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A121. Mud marks in trees near right bank, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A122. View of trailers from left bank to right bank, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A123. View of left bank high-water mark at trailer profile, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A124. Right bank high-water mark at trailer profile, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A125. Right bank high-water mark at restaurant trailer, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A126. View downstream toward lower slope-conveyance site, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A127. View downstream through cross-section 1, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A128. View downstream at right bank through cross section 1, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A129. View downstream through cross-section 3, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A130. View downstream at left bank through cross section 3, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A131. View downstream at right bank through cross-section 3, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A132. View downstream at upper slope-conveyance site, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A133. View upstream at lower slope-conveyance site, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A134. View upstream through cross-section 1, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A135. View upstream to left bank through cross-section 1, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A136. View upstream to right bank through cross-section 1, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A137. View upstream to left bank through cross-section 3, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A138. View upstream to right bank through cross-section 3, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A139. View upstream toward upper slope-conveyance site, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A140. High-water mark in parking lot on right bank, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A141. Mud on leaves near treetop, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

Figure A142. Mud on leaves of tree on right bank near trailers, Eldorado Canyon at Nelson Landing, Nevada, September 1974.

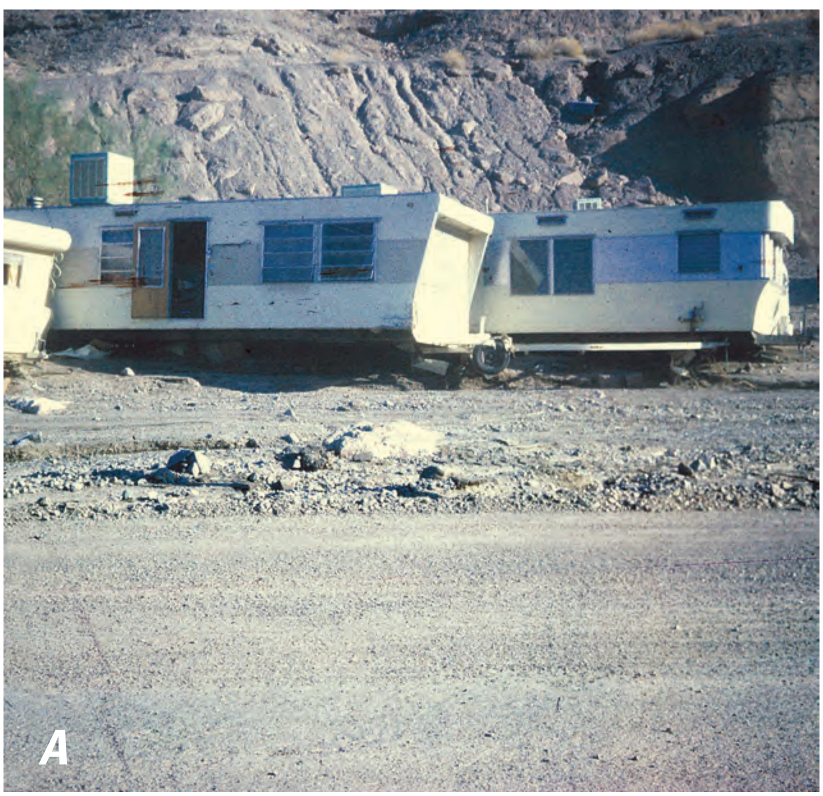

Figure A143. Damage to trailers (a), Eldorado Canyon at Nelson Landing, Nevada, September 1974.

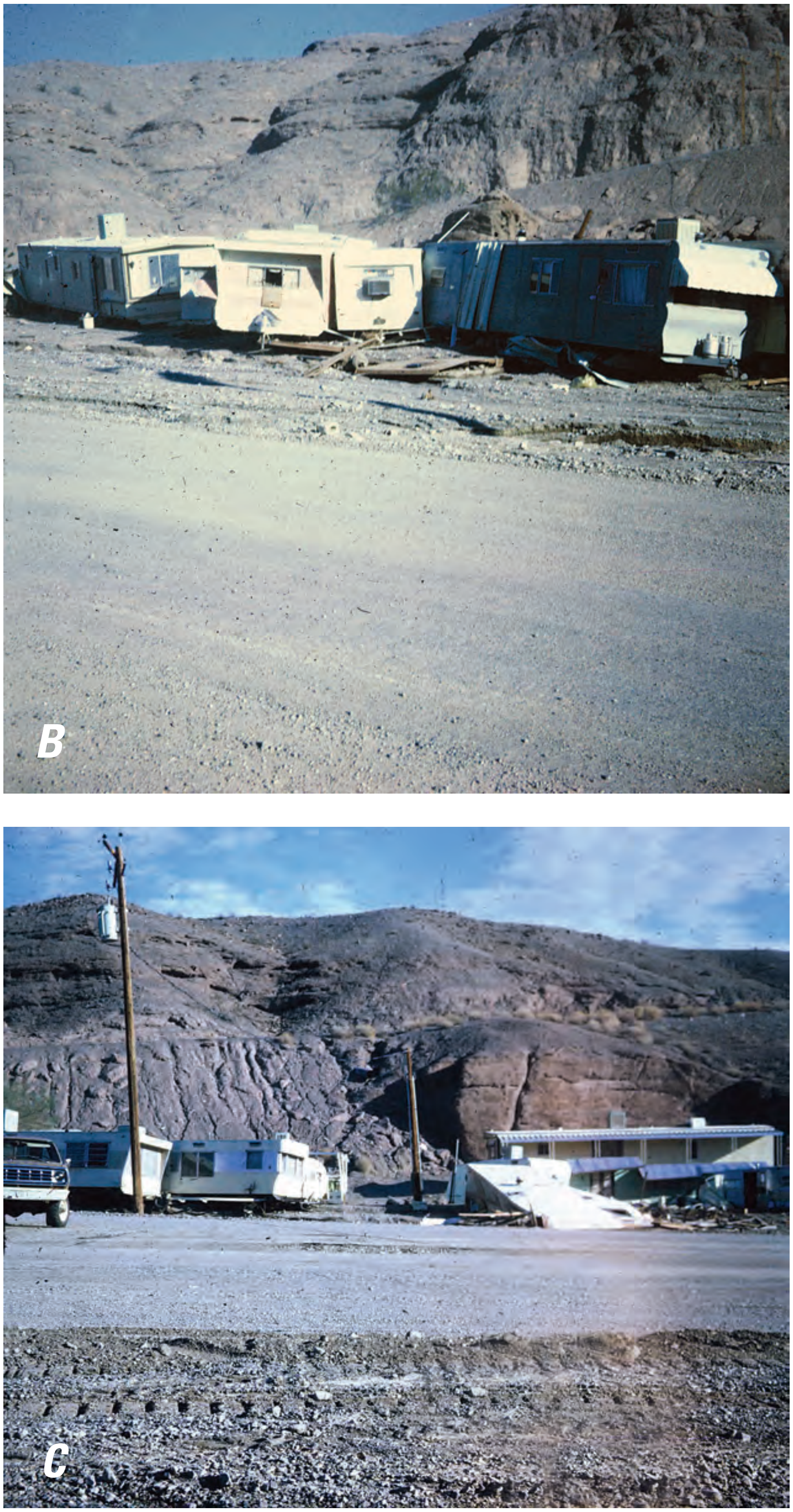

Figure A143. Damage to trailers (a), Eldorado Canyon at Nelson Landing, Nevada, September 1974.—Continued.

Figure A143. Damage to trailers (a), Eldorado Canyon at Nelson Landing, Nevada, September 1974.—Continued.

Figure A143. Damage to trailers (a), Eldorado Canyon at Nelson Landing, Nevada, September 1974.—Continued.

Figure A143. Damage to trailers (a), Eldorado Canyon at Nelson Landing, Nevada, September 1974.—Continued.

Figure A144. View looking upstream through slope-area reach, Eldorado Canyon at Nelson Landing, Nevada, August 28, 2003.

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Big Creek near Waynesville, North Carolina

(Also referred to as "near Sunburst," "Burnett Siding," and "above Lake Logan")

(Miscellaneous ungaged site, Big Creek basin, USGS North Carolina Water Science Center)

Review of peak discharge for the flood of August 30, 1940

Location: This flood was located about 1.6 mi northwest of Adako, N.C. at 35.9161N and 81.7292W.

Published peak discharge: A peak discharge of 13,000 ft^3/s is published in Crippen and Bue (1977). A peak discharge of 12,500 ft^3/s is published in Costa (1987a, 1987b). A peak discharge of 12,000 ft^3/s is published in U.S. Geological Survey (1949). The indirect measurement shows the computed and reviewed peak discharge as 12,400 ft^3/s.

Drainage area: The drainage area for this site varies by publication as follows:

Publication	Drainage area (mi^2)
Crippen and Bue, 1977	1.32
Costa, 1987a, 1987b	1.69 (4.38 km^2)
U.S. Geological Survey, 1949	1.69
Indirect measurement notes, 1941	1.69 (planimeter, unknown quad)
Topographic map (7.5 minute) estimate, 2003	1.93 (by planimeter)

The indirect measurement notes do not give a specific location of the surveyed site. The survey site is assumed to be about 700 ft upstream of the mouth. The indirect measurement review states that the Tennessee Valley Authority (TVA) made an indirect computation at a site 500 ft upstream of the USGS miscellaneous site and assigned a drainage area of 1.32 mi^2. This may explain the drainage area of 1.32 mi^2 given by Crippen and Bue (1977).

Data for storm causing flood: The TVA report "Floods of August 1940 in Tennessee River Basin" shows an average rainfall of 9.0 in. for the Big Creek basin. Individual rain gages in the area show rainfall amounts as much as about 12 in., over 20–40 hours. The main storm lasted about 22–27 hours.

U.S. Geological Survey (1949) refers to this storm as the "late-August storm," which was a comparatively local meteorological disturbance in the Little Tennessee and French Broad River basins. That report states that rainfall ranged from 8 to 13 in. for periods of 20 to 30 hours. In Haywood County, where Big Creek is located, published rainfall totals at 12 locations ranged from 3.5 to 11.3 in. Many of these values were obtained from a bucket survey and were furnished by the TVA. Historical photographs taken after the August 30, 1940, flood and during the 2003 review and described herein are provided in figures A145–A148.

Method of peak discharge determination: A three-section slope-area measurement was made on May 6, 1941, more than 8 months after the flood. There is no explanation for the time lapse between the flood and the survey. There is no indication that high-water marks were flagged soon after the flood or if they were located during the May 6 survey. A couple of marks are described as "good," which is hard to believe 8 months after the flood. The plotted high-water profile appears consistent with most marks lining up fairly well.

Another discrepancy is that the front sheet of the indirect measurement shows the date of the flood as August 30, 1941, rather than 1940. This probably is an inadvertent typographical error.

The actual location of the survey is assumed to be about 700 ft upstream of the mouth of Big Creek. The indirect measurement notes do not include a location description.

A number of manual computations originally were made using all three cross sections and also using only two sections. The two-section reach from the upstream to the middle section was expanding and was not used. The two-section reach from the middle to the downstream section was contracting and was used to compute a peak discharge of 12,400 ft^3/s. Although a number of other computations were tried, the discharge of 12,400 ft^3/s was the final discharge selected. This review revealed a minor error of about 5 percent in the cross-sectional area of the middle cross section, which probably has little effect on the final result.

For this review, all three cross sections, the original "*n*" values and the original water-surface elevations were entered in the slope-area computation (SAC) program. A peak discharge of 16,400 ft^3/s was computed using all three sections, but because of the expanding reach from sections 020 to 075, this computation is not acceptable. A peak discharge of 11,800 ft^3/s was computed using only the middle (075) and downstream (125) sections. This is 5 percent less than the original hand-computed discharge.

Froude numbers were not computed in the original hand computations. The SAC computations for the discharge of 11,800 ft^3/s gave Froude numbers of 1.0 for section 075, and 1.2 for section 125. Average velocities ranged from 17 ft/s at section 075 to 21 ft/s at section 125. The water surface fall was 3.55 ft in a distance of 50 ft (water slope = 0.071 ft/ft).

Possible sources of error: The most obvious and significant source of error for this indirect measurement is that it was most probably a debris flow/debris avalanche rather than a water flood. First hand reports, including field observations, notes and photographs document the mountain slides that occurred in the upstream reaches of Big Creek and the resultant scour and deposition of rocks, boulders, and sediment in the downstream reaches. A report, “Mountain Slides on the West Fork of the Pigeon River”, by the TVA (HD-1044, no date), provides a detailed description of the mountain slides in the Big Creek basin.

Photographs taken at or near the indirect measurement site show many large rocks and boulders in the channel. There is also evidence of significant scour of the banks, which contributes to uncertainty in cross-sectional area at the flood peak.

Another source of error is the delayed time (more than 8 months) between the flood and the indirect measurement survey. The accuracy of high-water marks is questionable. Very high velocities (20 ft/s or more) are indicated by the computations, and Froude numbers slightly exceed 1 (critical to supercritical flow). The reach length is only 50 ft.

Recommendations of what could have been done differently: The site should have been visited soon after the flood rather than 8 months later. This may have revealed more definitively that a debris flow occurred and that a standard indirect measurement would not be reliable. However, debris-flow processes were poorly understood in 1940; hence, recognition and identification of a debris flow likely would have been unrealistic. Photographs immediately after the flow would have been useful. There probably is no reliable way to determine the water discharge for this flood.

Site visit and review: The site was visited on August 25, 2003, by V.B. Sauer and Gene Barker (USGS). Although the exact location of the slope-area survey is uncertain, the channel near the slope-area survey (about 700 ft upstream of the mouth of Big Creek) is extremely overgrown with weeds, brush, and trees. The channel has a steep gradient (0.071 ft/ft) with large rocks and small boulders throughout. Photographs are included for the point where Big Creek enters West Fork Pigeon River, which shows a very rocky channel with large rocks along the right bank. A USGS gaging station on the right bank of West Fork Pigeon River, about 600 ft upstream of the mouth of Big Creek, has been operated since February 26, 1954. The station description for this gaging station does not mention the 1940 flood. The largest discharge for this site since 1954 is 9,740 ft^3/s. Drainage area is 27.6 mi^2.

Recommendations: The original peak discharge should not be used and should be removed from the record because the peak discharge is unreliable. However, the fact that a large and extraordinary flood occurred should be retained and documented in some way.

The peak discharge for this site is unreliable because of the very strong evidence that this was a debris flow and not a water flood. Conditions are such that it would be incorrect to recompute, or determine using other methods, a reliable peak discharge.

In addition, an indirect measurement for the August 1940 flood for a stream named “Big Branch” was found (but not reviewed for this study). This indirect measurement is named “Tributary to Little East Fork Pigeon River (near High Top) near Sunburst, N.C.” This was likely a debris flow as well on the basis of the geomorphic setting, and this indirect measurement should be reviewed.

Figure A145. Debris avalanche scar in headwaters of Big Creek, Big Creek near Waynesville, North Carolina, August 1940.

Figure A146. View looking downstream of slope-area reach, Big Creek near Waynesville, North Carolina, August 1940.

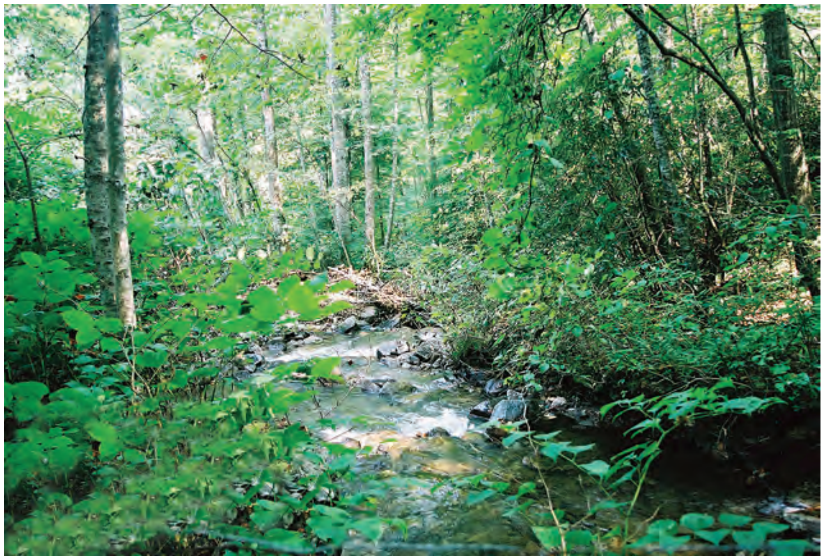

Figure A147. View looking downstream about 700 feet upstream of mouth, Big Creek near Waynesville, North Carolina, August 25, 2003.

Figure A148. View looking downstream along right bank opposite mouth of Big Creek, likely source of coarse boulders, West Fork Pigeon River near Waynesville, North Carolina, August 25, 2003.

Wilson Creek near Adako, North Carolina

(Miscellaneous ungaged site in the Yadkin basin,
USGS North Carolina Water Science Center)

Review of peak discharge for the flood of August 13, 1940

Location: This flood was located about 1.6 mi northwest of Adako, N.C. at 35.9161N and 81.7292W.

Published peak discharge: The peak discharge for the August 13, 1940, flood, as published in U.S. Geological Survey (1949), is 99,000 ft^3/s. The computed peak is described as "reliable" in the slope-area narrative statement, only identified by the initials H.J., written May 2, 1941.

An old gaging station, at latitude 35°55'10", longitude 81°44'00", as described in the 1950 compilation report (U.S. Geological Survey, unpub. data, 1950), was operated from 1921 to 1922. The field visit of August 25, 2003, places the indirect site at about 35°54'58", 81°43'45", by GPS, which is slightly downstream of the gaging station. However, there is no direct evidence that this is the correct location because the original field notes for the 1940 indirect measurement can not be located. The Collettsville 7.5-minute quadrangle was used for location purposes.

Remnants of an old low-water dam were located just upstream of Brown Mountain Beach at latitude 35°54'33", longitude 81°43'38" (GPS). This probably is the dam used by Granite Falls Manufacturing Co. (permittee in FPC Project No. 81). This dam is about 0.6 mi downstream of the probable location of the slope-area measurement. A statement in the compilation report for this reach of Wilson Creek indicates "uncomprehendable scour of the river bed and the valley hill sides." This would indicate that the dam may have been washed out by the 1940 flood, thus increasing the peak discharge and volume of the flood's capacity to exacerbate scour downstream of the dam.

Drainage area: A drainage area of 65.5 mi^2 for the 1921 gaging station was measured in 1924 on the 1905 Morganton quadrangle, scale 1:125,000. According to the 1950 compilation report (U.S. Geological Survey, unpub. data, 1950), the drainage area was rounded to 66 mi^2 for "published records," and this is the drainage area published in U.S. Geological Survey (1949). The drainage area may be in error because of the map scale used for the original determination. A recomputation of drainage area using 1:24,000 maps is advisable.

Data for storm causing flood: Two distinct, but separate storms occurred in August 1940. The first storm is commonly referred to as the mid-August storm, and is the storm that caused the peak discharge on Wilson Creek. This storm resulted from a hurricane in the Atlantic Ocean about August 8. Precipitation greater than 15 in. for the entire storm, and 8 in. for a single day was measured at numerous points in North Carolina. In Avery County, a total of 8.98 in. was recorded on August 13, and a 4-day storm total of 15 in. was recorded at the Crossnore station. In Caldwell County, two precipitation stations at Lenoir recorded storm totals of 8.8 and 11.1 in., respectively. The Wilson Creek basin covers much of both counties. A photograph taken during the 2003 review and described herein is provided in figure A149.

Method of peak discharge determination: The peak discharge for the August 13, 1940, flood was determined by a three-section slope-area measurement. The field notes and computations for this measurement can not be found, and therefore, a detailed review can not be made. A copy of the original review notes for the measurement is available. This review was made by someone with the initials "H.J.," and the review was made on May 2, 1941, more than 8 months after the flood.

The review notes by H.J. stated that a second slope-area measurement was attempted farther upstream but was not used because the roughness coefficients that were applied resulted in "excessive velocities and unreasonable discharges." The review also indicates there was "very excessive turbulence" in this reach. The writer also hints at the possibility that a critical depth computation may have been tried, but this is not certain, and H.J. did not give any results.

A rating-curve plot for the flow range greater than 10,000 ft^3/s was found. It was apparently developed for the 1950 compilation report (U.S. Geological Survey, unpub. data, 1950) to define the peak discharge for the 1916 flood, another very large flood for Wilson Creek. The station analysis indicates that this is a very large revision of the previous high-stage part of the rating curve. Other than the 1940 slope-area measurement, there are no other discharge measurements for definition of this rating. It appears to be a largely empirical or hand-drawn rating based on the 1940 indirect measurement and some type of velocity-area analysis for stages lower than the 1940 flood.

Possible sources of error: The fact that the original slope-area measurement has been lost and can not be reviewed in detail makes it difficult to determine possible sources of error. The field visit of August 25, 2003, to the probable location of the measurement did not reveal any obvious sources of error. The reach is straight for a long distance, with fairly steep banks. The streambed consists of gravel, large cobbles, and some small- to medium-size boulders.

The number and quality of high-water marks may be a source of uncertainty because it is not known exactly when the marks were flagged or when the slope-area measurement was made. If the field work was done soon after the flood, high-water marks may have been of good quality. However, the review was written more than 8 months after the flood, and the date of the flagging and (or) survey is not given, so the high-water marks are an uncertainty. The review does not mention high-water marks, so this omission may be an indication that the high-water marks were good.

In the extreme upstream reaches where the small tributaries come off the mountain slopes, some debris slides and flows may have occurred because they are common in this area. However, by the time the flood reached the site of the indirect measurement, most of the debris load should have been deposited. At the community of Mortimor, N.C., about 6 mi upstream of the slope-area site, buildings, roads, and railroads had been washed away during the flood. Old photographs posted in the Wilson Creek Visitors Center that were taken during the flood at Mortimor show flow and sedimentologic conditions that appear to be a water flow. There is no evidence of debris flows.

Recommendations of what could have been done differently: The main recommendation would be to properly preserve and archive all original surveys, measurements, field notes, and other material.

Site visit and review: A field visit to the probable site of the slope-area measurement and to several points upstream, including the Wilson Creek Wild and Scenic River Visitors Center, was made on August 25, 2003, by Vernon Sauer and Gene Barker (USGS Asheville Water Science Center field office). Photographs were taken at several places. Wilson Creek was declared a Wild and Scenic River in August 2000 by President Clinton. The visitors center contains information about the 1916 and 1940 floods, including photographs and various written and personal accounts.

Recommendations: The original peak discharge of 99,000 ft3/s should be accepted as published on the basis of the field inspection and the rating should be an “estimate.” The drainage area should be checked using 1:24,000-scale maps.

The writer (H.J.) of the original review of the indirect measurement states that the measurement results are reasonable and reliable. Because the measurement is not available at this time, there is nothing to contradict or support the reliability of the measurement.

Figure A149. View looking upstream of slope-area reach, Wilson Creek near Adako, North Carolina, August 25, 2003. No original photographs of 1940 flood were found.

El Rancho Arroyo near Pojoaque, New Mexico

(Miscellaneous ungaged site in the Rio Grande basin,
USGS New Mexico Water Science Center)

Review of peak discharge for the flood of August 22, 1952

Location: The flood was located about 3 mi west of Pojoaque, N.M. at 35.8902N and 106.0829W.

Published peak discharge: The peak discharge is 44,600 ft^3/s, August 22, 1952. This peak discharge was not published by the USGS at the time. After review at USGS Headquarters, the consensus was that the discharge was too uncertain to publish. However, Tate Dalrymple included the peak discharge in Chow's Handbook of Applied Hydrology (Chow, 1964, p. 25–12). The peak resurfaced during the nationwide flood-frequency project (Patterson, 1965).

From September 17, 1963, memorandum from Wilber Heckler, New Mexico District Engineer, to the Chief, Basic Records Section, Washington, D.C.:

> "The results of this measurement have never been published. Computations were reviewed by Dalrymple, Benson, and Hulsing, and apparently there was sufficient doubt about the result to decide against publishing it. The measurement was reviewed by Patterson September 17, 1963, and discussed with engineers in the Santa Fe office. Doubt still exists on portions of the measurement such as its warped section, high-water line on one side considerably higher on one side than the other, up to 2 ½ feet of scour in part of the channel, all vegetation in channel did not wash out despite the high velocities."

> "One conclusion is to recommend that the results of this measurement should not be published. We agree it was an unusual flood and suggest consideration be given to making mention of it in the list of miscellaneous measurements, but not to publish a discharge figure."

The discharge (44,000 ft^3/s from a drainage area of 6.7 mi^2) was included in Crippen and Bue (1977); they evidently took the value from Chow (1964). The discharge (45,000 ft^3/s from a drainage area of 6.9 mi^2) also appears in Glancy and Harmsen (1975, table 3) but is mislabeled as Trujillo Arroyo near Hillsboro, N. Mex. This is documented in a memorandum from A.G. Scott to P.A. Glancy dated October 24, 1975, and the November 5, 1975, response from J.P. Monis.

The discharge also was the topic of an exchange of memoranda from W.W. Reedy, Bureau of Reclamation, to William Hale, USGS (May 10, 1977), and a response from R.P. Thomas for Mr. Hale (May 16, 1977). The gist of these exchanges was that USGS had never published the discharge of 44,600 ft^3/s because of concern about the unusual hydraulic conditions and continued to believe the peak discharge value was too large.

Drainage area: 6.7 mi^2. The map scale for the original determination is unknown, but A.G. Scott (USGS) in a February 17, 1972, memorandum to M.S. Petersen (USGS) noted that he had

> "… checked the drainage area on 7 ½ min quads and arrived at 6.82 sq. mi."

Scott Waltemeyer (USGS New Mexico Water Science Center) used the 30-m (NED) and GIS to compute a drainage area of 6.773 mi^2 as part of this 2003 review.

Data for storm causing flood: The following is extracted from the September 22, 1952, summary prepared by Hugh Hudson (USGS):

> "State road 4, connecting Los Alamos with Santa Fe, was impassable for several hours on the evening of Aug. 22 as a result of extremely heavy rain in the headwaters of El Rancho and adjacent arroyos. El Rancho Arroyo crosses state road 4 as three arroyos which merge about 1,000 feet below the highway and about 300 feet above this slope-area reach. The Soil Conservation Service made a limited bucket survey after the storm and found that the rainfall in not more than an hour was 5 inches at El Rancho. Indications are that El Rancho was not in the center of the storm. The headwater drainage is uninhabited, so no rainfall data are obtainable where the maximum rainfall apparently occurred."

The following undated handwritten note was added by Hugh Hudson:

> "According to local residents, this flood is comparable only to the flood of 1829, and may have exceeded the 1829 flood."

Photographs taken during the 2003 review and described herein are provided in figures A150–A155.

Method of peak discharge determination: The peak discharge is based on a three-section slope-area measurement. The measurement had a number of nonstandard conditions as noted in section "Possible sources of error." In response to preliminary reviews, the third section was analyzed, and the cross sections were probed. The reach was slightly contracting with high velocities; the resulting Froude numbers were high—1.63, 1.61, and 1.83.

This survey was conducted by Hugh Hudson, and the results were reviewed by W.P. Somers, Tate Dalrymple, H. Hulsing, and M.A. Benson (USGS). It is difficult to conceive of a more qualified set of flood specialists. The review by the latter three persons included a field inspection on March 30, 1953. Their summary review comments (April 15, 1953) conclude with,

> "It is felt that unless additional field data show otherwise that this figure is the best obtainable, and it is recommended that it be checked and used."

The May 16, 1977, memorandum from R.P. Thomas to the Bureau of Reclamation ends with,

> "Also, a 1963 flood-routing analysis, using records at a regular station on the Rio Grande about 3 miles downstream, indicated 44,600 ft^3/sec to be too high."

That analysis apparently is described in a September 30, 1963, memorandum from W.L. Heckler (USGS) to the Basic Records Section in Washington, D.C. Flow at the gaging station on the Rio Grande at Otowi increased from 2,000 to 7,000 ft^3/s and receded in 2.5 hours. That represented a storm runoff of about 1,000 acre-ft at Otowi. The memorandum notes that a flood duration of 0.5 hour at El Rancho Arroyo with a peak discharge of 44,600 ft^3/s would produce 900 acre-ft from that arroyo alone (assuming a triangular hydrograph), and other tributaries were known to have carried some flow. The 2003 review notes that this is a mass balance rather than a routing analysis; such an analysis would have to consider the effects of bank storage when a very sharp peak discharge occurs in a wide, normally dry channel. A true routing would reduce the 900 acre-ft contribution from El Rancho by the time it reaches Otowi. This would allow a contribution from other tributaries.

Possible sources of error: The possible sources of error are well documented in the earlier reviews. They include:

- A transverse change in elevation from right to left of 6.3 ft at section A, 4.2 ft at section B, and 2.8 ft at section C. Top widths are slightly greater than 300 ft at all three sections. Longitudinal fall is 5.35 ft in 275 ft (slope = 0.019 ft/ft).

- Several irregularities were noted in the water-surface profile on the right bank.

- Computations were based on the then recommended practice of probing cross sections to determine scour depths. Those probed depths were included in the cross-sectional properties. The probed depths increase the cross-sectional areas by about 15–20 percent.

- High velocities (about 25 ft/s) lead to velocity heads of about 10 ft and Froude numbers of about 1.5–1.6, which are high but not unprecedented.

As part of the 2003 review, the original computations were coded for the current slope-area computation (SAC) program. When the water surface is treated like it was in the 1952 computation, SAC produces a result of 44,500 ft^3/s, agreeing with the original computation. Rerunning the SAC excluding the probed depths gives a discharge of 34,800 ft^3/s. Froude numbers excluding the probed depths were still high (1.78, 1.46, and 1.53), and the reach expands from section A to section B. However, the difference between 0- and 100-percent energy recovery for the three-section result is only 4 percent (for example, the expansion losses are accounted for properly and do not reduce the reliability of the measurement).

In the 1950s, the probing of depths was recommended. Currently (2007), that practice is not recommended unless there is strong evidence to support the idea that the channel filled after the peak discharge. The opposite is true in this case. The notes and reviews acknowledge that vegetation was protruding from the bed—a strong indicator that the amount of new deposition was small. However, with the sand beds that are common in New Mexico, one can almost always get penetration with a probe, which increases the cross-sectional area. Given the high velocities, this added area increases the discharge by significant amounts.

Recommendations of what could have been done differently: Everything was done according to proper hydraulic methods of the time, including some of the most extensive reviews imaginable. However, given the evidence of rooted vegetation protruding from the bed, probing the bed was definitely a questionable practice. In addition, the nearly direct link between the superelevation, the channel alignment, and the high velocities should have been recognized.

Site visit and review: The site was visited on August 5, 2003, by John Costa (USGS Office of Surface Water), Scott Waltemeyer (USGS New Mexico Water Science Center), Mark Smith (USGS Central Region), and Kenneth Wahl (USGS retired).

The site looks remarkably similar to the photographs taken in 1952, including a sand/gravel bed with small tufts of vegetation protruding. In viewing the reach as a whole, the channel alignment is slightly curving to the left throughout, and the right bank is largely a bluff. This alignment, coupled with high velocities (in the range of 20 ft/s), could explain the superelevation on the right bank; a velocity of 20 ft/s produces a potential static head of 6.2 ft.

Recommendations: The original peak discharge of 44,600 ft^3/s should not be used and should be retained in the peak-flow data base. The rating should be no better than “poor.”

The original peak discharge is overestimated because of the inclusion of probed depths in the flow cross section. The computation can be easily corrected by recomputing the measurement using the original parameters and using the actual surveyed cross sections. This will result in a revised discharge of 34,800 ft^3/s and a unit runoff of 5,200 $(ft^3/s)/mi^2$.

Questions undoubtedly will remain, and the result can be considered no better than poor for all the reasons used originally to withhold publication.

Figure A150. View looking downstream of cross section 1, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

Figure A151. View looking upstream toward cross section 1, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

Figure A152. Right bank in slope-area reach with flood-scoured sandstone, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

Figure A153. View downstream toward right bank, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

Figure A154. View to right bank at new cross-section 3, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

Figure A155. View toward left bank at new cross section 3, El Rancho Arroyo near Pojoaque, New Mexico, August 5, 2003.

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Cimarron Creek Tributary near Cimarron, New Mexico

(Miscellaneous ungaged site in the Canadian River basin, USGS New Mexico Water Science Center)

Review of peak discharge for the flood of June 5, 1958

Location: The flood occurred at a culvert on U.S. Highway 64, about 2 mi west of Cimarron, N.M. at 36.51919N and 104.95492W.

Published peak discharge: The published peak discharge determined from the indirect culvert measurement is 337 ft^3/s on June 5, 1958. The measurement was rated fair.

Drainage area: The drainage area listed for the 1958 determination was “about 0.05 mi^2.” The area was determined by planimeter from the 1955 Cimarron quadrangle with a scale of 1:62,500 and a contour interval of 40 ft. Scott Waltemeyer (USGS New Mexico Water Science Center) used the 30-m NED and GIS to recompute a drainage area of 0.15 mi^2 as part of the 2003 review.

Data for storm causing flood: There is no information on the storm that caused the flood. The measurement summary notes only the location, type of computation, and result. The flooding undoubtedly was the result of a small, intense thunderstorm cell common in this part of New Mexico. A photograph taken during the 2003 review and described herein is provided in figure A156.

Method of peak discharge determination: The peak discharge is based on type I flow (inlet control) in a skewed concrete box culvert. The measurement was reviewed at headquarters by Harry Barnes, Jr. (USGS) on July 28, 1960.

The survey and computation were straightforward and were done correctly. The bed at the approach section had fresh sand fill (about 2 ft). This fill was assumed to have occurred after the peak discharge.

As part of the 2003 review, the computations were coded and run through the current USGS culvert analysis program (CAP). Those results confirm the original result of 337 ft^3/s and type I flow.

Possible sources of error: The most likely sources of error in the measurement are: (1) the normal assumption that the culvert is free from debris and obstructions, (2) the assumption that the fill in the approach occurred after the peak discharge, and (3) the size of the basin that produced the flood.

Recommendations of what could have been done differently: There should have been a discussion of the rainfall that produced the flood. Those details are nearly impossible to determine years after the fact for small-scale floods in sparsely populated areas. The erroneous drainage area is the other shortcoming of this measurement, and was nearly impossible to correct given the maps and technology available at the time. The only recourse would have been to define the perimeter of the basin with a field survey—a monumental undertaking in so rugged a basin.

Site visit and review: The site was visited on August 5, 2003, by John Costa (USGS Office of Surface Water), Scott Waltemeyer (USGS New Mexico Water Science Center), Mark Smith (USGS Central Region), and Kenneth Wahl (USGS retired). The highway has been widened since 1958, and the culvert entrances have been changed to accommodate the wider roadway. However, the skewed culvert barrel is still in place. There was no evidence that past flows had been other than water floods that carry large amounts of sediment as bed load.

Debris blocking the culvert entrance is unlikely given the lack of evidence of blockage in photographs taken during the survey. However, the fill observed in the approach at the time of the survey possibly existed during the flood given the high rates of sand transport. The cross-sectional area computed for the approach was about 200 ft^2, of which about 35–40 ft^2 was filled with sand. Even if the sand had been present at the peak discharge, flow would have been subcritical in the approach, and the culvert inlet would have been the control (type I flow). Therefore, the discharge would have been essentially as originally computed.

Recommendations: The original peak discharge of 337 ft^3/s should be accepted as published, but rounded to 340 ft^3/s. The drainage area should be corrected to 0.15 mi^2, and the numbers updated in future references to this flood.

There is no doubt that the rain and runoff from this small basin and from the surrounding area was exceptional. The computations are done correctly. The drainage area based on the 1:62,500-scale map was only one-third that determined by 2003 maps.

Figure A156. Site of culvert measurement, Cimarron Creek Tributary near Cimarron, New Mexico, August 5, 2003.

Meyers Canyon near Mitchell, Oregon

(Miscellaneous ungaged site in the add John Day River basin, USGS Oregon Water Science Center)

Review of peak discharge for flood of July 13, 1956

Location: The flood occurred about 4.1 mi northwest of Mitchell, Oreg., at 44.61995N and 120.19672W.

Published peak discharge: The published peak discharge for this flood is 54,500 ft^3/s and was rated fair.

Drainage area: The 12.7 mi^2 drainage basin is steep and sparsely vegetated. The soil is hydrophobic and easily eroded. The basin is about 6 mi long and 3.5 mi wide. The channel in the downstream part of the basin is deeply eroded into valley alluvium and forms a sinuous and deep (about 20–30 ft) canyon.

Data for storm causing flood: An intense convection storm produced excessive rainfall and caused severe flooding in tributaries to Bridge Creek near Mitchell, Oreg. Meyers Canyon, about 6 mi northwest of Mitchell, Oreg., was the hardest hit of these drainages. The storm reportedly centered over this basin and produced record runoff. There is continuing controversy among engineers and hydrologists who have studied this flood primarily because it is difficult to imagine that this extraordinary flood did not leave a geomorphic record in Bridge Creek. This may be because peak discharge only lasted a few minutes and the total volume of runoff was small; thus, the flood wave rapidly attenuated.

W.D. Wilkinson, an Oregon State College geology professor, was camped along Service Creek Road in the upstream part of Meyers Canyon basin during the flood. He reported rainfall starting about 4:30 p.m. on July 13, 1956, and increasing in intensity until about 5 p.m. The first flood crest passed his camp at about 5:15 p.m. and crested at about 7–8 ft. A second crest passed about 6:10 p.m. but was lower, about 4–5 ft. The storm was intense until about 6 p.m. and diminished until the rain stopped at 7 p.m. The most intense rainfall lasted only about 30 minutes (between 4:30 and 5 p.m.). Mr. Wilkinson observed sheet runoff at the base of the hills as deep as 2 in. Velocity of the 2.5-ft deep overbank flow near his camp was high enough to move his Travelall truck 500 ft downstream. There were no direct measurements of rainfall in the upstream part of the basin. Bucket surveys in Mitchell and at Girds Creek produced estimates in the 3.5- to 4-in. range; maximum rainfall amounts and intensities were likely greater but are unknown. Historical photographs taken after the flood of July 13, 1956, and photographs taken during the 2003 review and described herein are provided in figures A157–A171.

Method of peak discharge determination: A three-section slope-area measurement was made in a steep, narrow, gully 0.3 mi upstream of the mouth of Meyers Canyon. There was so much expansion between sections A and B that a three-section solution could not be obtained. The flow estimate is from a two-section slope-area measurement. The 30-ft deep gully appeared to have been overtopped leaving a line of good to fair high-water marks along the margin of the 400-ft wide valley floor. The left-bank marks were taken from the stiff stems of sagebrush that covered the left-bank overflow. High-water marks along the right bank primarily were fine debris on the ground at the gently sloping edge of the grassy overflow area. Marks along this bank are superelevated from a combination of an upstream breakout and a small rounded ridge perpendicular to the channel downstream.

The effect of the upstream breakout causes the major controversy surrounding this flow estimate. This “disagreement” has lasted for 50 years. The argument is that the high-water definition used for the slope-area computation does not represent the elevation of water in the main channel, and flow was small enough to be contained within the channel.

The channel is forced into an “S” configuration by projecting side ridges on each bank just upstream of the slope-area reach. The channel upstream of these side ridges is straighter and much larger in cross section. This reach of channel was not used for the slope-area measurement because no high-water marks could be found. Flow probably was contained in the channel, but the nearly vertical, unvegetated sidewalls did not trap any debris or erode enough to leave a peak-stage record. The right-bank breakout occurred just downstream of the S-shaped channel, and evidence is still visible. A breakout on the left bank that is not as obvious would have affected high-flow definition along that bank. Marks along the right bank are about 2.8 ft higher than those on the left bank at section A, about 5.5 ft higher at section B, and about 1.7 ft higher at section C. Some of this difference is attributed to channel curvature, but most may be caused by the upstream breakout. High-water slope along the right bank is almost flat between sections A and B. The high-water profiles from sections B to C are steep (more than 8 ft in about 220 ft) (slope = 0.036 ft/ft). Fall on the left bank in this reach is 6.75 ft, and fall along the right bank is 10.50 ft. The left-bank high-water marks define flow about 3 ft deep at the edge of the main channel at section B and about 3.5 ft deep at the channels edge at section C. If the high-water marks define a flow connected from valley margin to valley margin, there is sufficient area to carry the computed discharge at velocities less than 30 ft/s. Froude numbers ranged from 1.06 to 1.25, which are high but not unprecedented. Channel curvature and channel alignment made it difficult to locate the cross sections perpendicular to the flow.

Hydrologists from the Bureau of Reclamation have never agreed with the discharge computed from this slope-area measurement. Francis Hart and G.W. Kirkpatrick of the Bureau of Reclamation did a reconnaissance of the flood-affected area on August 13–14, 1956. They recorded few actual data but made several observations. They point out an area of erosion on the right rim of the channel between sections B and C caused by overbank return flow. They also assumed left-bank overflow returned to the main channel through a small draw between these two sections. This observation is not supported by the left-bank high-water marks nor do the right-bank marks support the theory that all or most of the right-bank overflow returned to the channel at the point defined by erosion on the channel rim. A Bureau of Reclamation report dated November 23, 1956, is included in the file and summarized these observations. It includes copies of photographs supporting the observations.

On August 9, 1956, Harry Hulsing (USGS) visited the site to review the assigned "*n*" values and made no comment about the possibility of the peak flow not being connected across the channel. Roughness coefficients were raised from 0.045 to 0.050 (increase of 11 percent) for the main channel to compensate for channel irregularities. This change reduced the computed discharge from 64,000 to 54,500 ft^3/s. He returned with G.L. Bodhaine (USGS) on October 22–23, 1956, in response to Mr. Hart's observations. They found no reason to discredit the results of the slope-area measurement. They were convinced that overbank flow was connected to flow in the main channel at the time of the peak discharge. During his first visit, Harry Hulsing investigated the upstream part of the basin and described a 2-mi stretch of the Service Creek Road as one long debris pile. He described all the side slopes as deeply gullied and commented that all the culverts were washed out, buried, or clogged with debris.

Bridge Creek runs through Mitchell and has a history of major flooding. Flow estimates in Bridge Creek at Mitchell and from a slope-area measurement about 10 mi downstream of the mouth of Meyers Canyon are both about 14,400 ft^3/s. The Bureau of Reclamation contends that flow in Bridge Creek downstream of Meyers Canyon should have been far greater if the discharge computed for this flood is correct. USGS noted that flow volume from Meyers Canyon was not great, and flow was attenuated along the Bridge Creek Valley. There is no comment on the timing of the two peak discharges.

The Bureau of Reclamation has studied this flood as part of a project for spillway design in Central Oregon. They mapped 1,600 ft of channel in the area of the slope-area reach and ran a step-backwater model through the downstream 300 ft to try to estimate maximum possible discharge. Their results are on the order of 17,700 ft^3/s, about one-third of the USGS estimate. Their study approach and results are published in Levish and Ostenaa (1996).

Possible sources of error: Application of a two-section slope-area computation introduces the possibility for significant error. If the high-water marks do not define a continuous water surface, the computation is invalid. This could have been confirmed by obtaining high-water marks on the sagebrush near the edge of the channel or on the overflow plain. This is a highly erodible basin, and flows could have been hyperconcentrated. Flows probably were multidimensional and could have been unstable. The cross sections do not appear to be perpendicular to the flow, so cross-sectional area may be incorrect. The estimated rainfall amounts do not seem to support the peak discharge estimated for this flood.

Recommendations of what could have been done differently: In 1956, not much could have been done differently. High-water marks could have been surveyed near the main channel to verify a continuous water surface. There may have been evidence on some stiff sagebrush that was recoverable. A cross section could have been surveyed across the larger channel upstream of the breakout to estimate if the peak discharge could be contained in that channel. Evidently there were no high-water marks indicating flow outside the channel and no recoverable marks in the channel.

Site visit and review: The site was visited on April 4–5, 2003, by John Costa (USGS Office of Surface Water), Bob Jarrett (USGS Office of Surface Water), Mike Nolan (USGS Regional Specialist), and Glenn Hess and Jim O'Conner (USGS Oregon Water Science Center), John England (Bureau of Reclamation), Joe Weber (Federal Emergency Management Agency), and Gary Gallino (USGS retired). The field-review team inspected flood remnants in the upstream part of the basin as well as the downstream reach. There were several buried trees near the mouth of the main canyon, and the mouths of tributary canyons provided evidence of extensive erosion, sediment transport, and deposition.

There was discussion of the benefits of using a two-dimensional model through the reach to try to simulate multidimensional flow. However, there is insufficient data, particularly water-surface elevations in the main channel, to justify the effort and improve the discharge. For future floods that have these types of unique hydraulic conditions, use of two-dimensional modeling and collection of appropriate site data should be encouraged. Evidence of the upstream breakout and the area of suspected return flow were inspected and discussed.

Recommendations: The original peak discharge of 54,500 ft^3/s should be accepted as published and the rating should be downgraded to "estimate."

The discharge estimate is so uncertain that its value should be viewed with great suspicion with respect to any determination of flood risk in other basins.

Figure A157. View looking downstream of slope-area reach, Meyers Canyon near Mitchell, Oregon, July 1956. Man standing at cross section B.

Figure A158. View of right-bank overflow, Meyers Canyon near Mitchell, Oregon, July 1956. Man standing at cross section B.

Figure A159. View looking downstream from above cross section B on right bank, Meyers Canyon near Mitchell, Oregon, July 1956.

Figure A160. View looking downstream of cross section B to cross section C, Meyers Canyon near Mitchell, Oregon, July 1956.

Figure A161. View looking upstream from downstream from cross section C, Meyers Canyon near Mitchell, Oregon, July 1956. Man standing at cross section C.

Figure A162. View looking upstream, Meyers Canyon near Mitchell, Oregon, July 1956. Man standing on right bank at cross section C.

Figure A163. View looking from right to left bank, Meyers Canyon near Mitchell, Oregon, July 1956. Man standing at cross section C.

Figure A164. View looking upstream to cross section B, Meyers Canyon near Mitchell, Oregon, July 1956.

Figure A165. View looking upstream toward slope-area reach, Meyers Canyon near Mitchell, Oregon, April 22, 2003.

Figure A166. View looking upstream from right-bank hillslope across to slope-area reach, Meyers Canyon near Mitchell, Oregon, April 21, 2003.

Figure A167. View looking downstream from area on right bank where flow likely broke out of canyon, Meyers Canyon near Mitchell, Oregon, April 21, 2003.

Figure A168. View from right bank looking across canyon toward possible return-flow gully that channeled flood-plain overflow back into canyon, Meyers Canyon near Mitchell, Oregon, April 21, 2003.

Figure A169. View looking downstream from left bank toward right bank and people standing in slope-area reach, Meyers Canyon near Mitchell, Oregon, April 21, 2003.

Figure A170. View looking downstream into main canyon toward slope-area reach, Meyers Canyon near Mitchell, Oregon, April 21, 2003.

Figure A171. Overland flow following rainfall in upstream part of Meyers Canyon Basin, Oregon, April 21, 2003. Almost no infiltration into fine-grained surficial material.

Lane Canyon near Nolin, Oregon

(Miscellaneous ungaged site in the Umatilla River basin,
USGS Oregon Water Science Center)

Review of peak discharge for the flood of July 26, 1965

Location: This flood was located about 1 mi southeast of Nolin, Ore., at 45.6729N and 119.0818W.

Published peak discharge: The published peak discharge for this flood is 28,500 ft^3/s and was rated fair.

Drainage area: The drainage area is 5.04 mi^2 and is characterized by steep, rocky, sparsely vegetated, erosive side slopes and a steep-gradient channel. The main channel and tributary canyons have numerous reworked fluvial deposits, but the review team agreed that debris flows were unlikely. The storm was so intense and the slopes so steep that infiltration probably was minimal.

Data for storm causing flood: A severe "cloudburst" dropped record precipitation over several small drainage basins in the Umatilla River basin in north-central Oregon on July 26, 1965. The State was just recovering from the devastating Christmas flood of December 1964 and January 1965 that destroyed much of the hydrologic data network in areas west of the Cascades Range (Waananen and others, 1971). The July storm covered the entire Lane Canyon basin according to local residents. Spear Canyon, a larger basin about 3 mi west of Lane Canyon, also was hit hard, but only part of the basin was affected by the most severe part of the storm. A house near the mouth of Spear Canyon was washed away by the flood, and one family member drowned. The storm was intense and short in duration. The most intense concentration of rain and hail lasted about 30 minutes (according to Mrs. Stroughton, a long-time local resident). She is sure no storm or flood of this magnitude had occurred in Lane Canyon during the 100+ years her family had lived in the area. (Mrs. Stroughton is a descendent of the Lane family for which the canyon is named.) The flood destroyed a county road bridge and a railroad bridge at the mouth of the canyon. This area is only about 30 mi north of Heppner where the worst flood disaster in Oregon history occurred in 1903. Historical photographs taken after the flood of July 26, 1965, and photographs taken during the 2003 review and described herein are provided in figures A172–A178.

Method of peak discharge determination: A two-section slope-area measurement was made in a good reach near the mouth of the canyon on August 17, 1965. The 300-ft reach length was limited by tributary inflow at the upstream end and by backwater from a bridge and road fill at the downstream end. Flow was contained in a straight, steep-gradient, contracting channel. The fall was 3.77 ft between the cross sections (slope = 0.013 ft/ft). The high-water profile was defined by good quality high-water marks on both banks. There was no significant difference in elevation on either bank.

The main channel is described as "hard pan" with some cobbles and exposed bedrock. The channel upstream of the reach is littered with cobbles, but the high flow swept the slope-area reach clean. At the time of the survey, scour in the reach seemed minimal. The flood likely carried a significant percentage of suspended sediment and may have been a hyperconcentrated flow. This sediment was deposited on the Umatilla River flood plain. Records of the amount of deposition probably could be recovered from the railroad because of the extensive cleanup and repair needed for the railroad bridge.

The Manning's "*n*" value of 0.032 seems reasonable. If the channel was not so steep-gradient, a smaller roughness coefficient may have been applicable. Manning's "*n*" for the "hard pan" bed and grassy side slopes was estimated at 0.025 but was increased to 0.032 because of some channel irregularities. Subdivision was considered for both sections during review but was deemed unnecessary. There were no significant overflow areas at either cross section.

The computed average velocity was 27–28 ft/s. Flow was supercritical throughout the reach with Froude numbers in the 1.8 to 1.9 range. This flood probably was a flow "spike" where gradually varied, one-dimensional flow was short lived if it occurred at all.

The Bureau of Reclamation reviewed this indirect measurement and the resulting discharge as part of their project on spillway design for projects in Central Oregon. They developed a topographic map and cross sections for the 400 m of stream channel immediately upstream of the mouth of the canyon and ran a step-backwater analysis. The results indicate 11,000 ft^3/s as a maximum discharge for a "clearwater" flood (Levish and Ostenaa, 1996).

Possible sources of error: The most probable source of error is using a two-section slope-area measurement. There is plenty of slope for another section, and (or) flow in the tributary canyon could have been measured and the reach extended upstream. Timing of peak flow from the tributary is not a problem for storms that are this short in duration.

The flood transported a lot of sediment. The deposits at the mouth of the canyon appear to be extensive.

Sediment concentration is not known but could be in the hyperconcentrated range. Flow may not have been gradually varied as required for the slope-area solution. Selecting "*n*" values for channels this steep is difficult. There is no verification that is universally accepted.

Recommendations of what could have been done differently: Another section could have been surveyed between sections 1 and 2. There is plenty of slope, and the sections could have been located a short distance upstream and downstream to allow for a third section. The reach could have been extended upstream by estimating or measuring tributary inflow that was a small percentage of the total discharge. Had this been done, several more sections could have been included in the computation.

Site visit and review: The site was visited on April 22–23, 2003, by John Costa (USGS Office of Surface Water), Mike Nolan (USGS Western Region Surface-Water Specialist), Bob Jarrett (USGS Office of Surface Water), and Glenn Hess and Jim O'Conner (USGS Oregon Water Science Center), John England (Bureau of Reclamation), Joe Weber (Federal Emergency Management Agency), and Gary Gallino (USGS, retired). The field team studied the downstream one-half of the basin looking for evidence of debris-flow deposits. The team concluded that the deposits were reworked alluvium and that the flood was not a debris flow. Some lobes suspected to be debris-flow deposits were observed at the mouth of some small tributaries, but they did not reach the main channel.

Recommendation: The original peak discharge of 28,500 ft^3/s should be accepted as published and the rating should be downgraded to "poor" because of the two-section solution, high Froude numbers, and large amounts of sediment.

The reach is an excellent slope-area site and application of the slope-area technique is appropriate.

Figure A172. View looking across Lane Canyon toward Umatilla River Valley, Lane Canyon near Nolin, Oregon, August 1965.

Figure A173. View looking downstream from upstream of cross section 1, Lane Canyon near Nolin, Oregon, August 1965.

Figure A174. View looking downstream toward mouth of canyon from downstream of slope-area reach, Lane Canyon near Nolin, Oregon, August 1965.

Figure A175. View of right bank from upstream of section 1, Lane Canyon near Nolin, Oregon, August 1965. Man is standing at section 1.

Figure A176. View looking downstream to left bank with sections 1 and 2 marked with orange signs, Lane Canyon near Nolin, Oregon, August 1965.

Figure A177. View looking upstream through slope-area reach, Lane Canyon near Nolin, Oregon, April 23, 2003.

Figure A178. View looking downstream through slope-area reach, Lane Canyon near Nolin, Oregon, April 23, 2003.

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Bronco Creek near Wikieup, Arizona

(Miscellaneous ungaged site in the Big Sandy basin, USGS Arizona Water Science Center)

Review of peak discharge for the flood of August 19, 1971

Location: This flood was located about 44 mi southeast of Kingman, Ariz., at 34.6764N and 113.5958W.

Published peak discharge: The published peak discharge for this flood is 73,500 ft^3/s and is rated poor. Other published discharge estimates are:

Publication	Discharge (ft^3/s)
Carmody (1980)	28,100
House and Pearthree (1995)	28,300
Hjalmarson and Phillips (1997)	96,800

Drainage area: 19 mi^2. The basin has three subbasins—Bronco Creek, Bronco Wash, and Greenwood Wash (so named in House and Pearthree, 1995).

Data for storm causing flood: A flashflood hit the Bronco Creek basin on August 19, 1971. About 3 in. of rain were measured in 45 minutes in Wikieup, Ariz., about 3 mi from the slope-area reach. This measurement seems to be the only local precipitation data for this storm. This flood is described as "virtually the largest known rainfall generated flood to come from a 50 square kilometer basin." Several investigators have attempted to analyze this flood using a variety of hydraulic and meteorological methods and have obtained a wide range of results. Historical photographs taken after the flood of August 19, 1971, and photographs taken during the 2003 review and described herein are provided in figures A179–A182.

Method of peak discharge determination: The original peak discharge estimate of 96,700 ft^3/s is based on a four-section slope-area measurement for a uniform reach. The discharge was reduced to 73,500 ft^3/s after roughness coefficients were increased during measurement review. The site was selected, and high-water marks flagged by H.W. Hjalmarson (USGS) on August 24, 1971. High-water marks and cross sections were surveyed on August 31, 1971. Byron Aldridge (USGS) did a contracted-opening computation for flow through the U.S. Highway 93 bridge to try to verify the computed discharge. Additional discharge estimates were made by several investigators using hydrometeorological methods, paleoflood techniques, and translatory wave theory. The original slope-area and the paleoflood (step-backwater) estimates probably are the most defensible. The occurrence of a series of large waves breaking over the highway bridge is supported by an eyewitness.

The original slope-area survey and bridge-contraction notes can not be found, but copies of the computations and review comments were available for use in this review. The slope-area reach is about 900-ft long and 400–500-ft wide and ends about 900 ft upstream of the U.S. Highway 93 bridge. The channel is an alluvial sand-bed channel. The original discharge computed by slope-area methods was reduced to 73,500 ft^3/s after roughness coefficients were increased from 0.030 to 0.040. Froude numbers ranged from 1.34 to 1.88. Tom Maddox (USGS), estimated velocities in the 25-ft/s range for the sediment sizes found in the slope-area reach. Velocity computed from the 73,500 ft^3/s discharge and the average 2.700-ft^2 cross-sectional area is about 27 ft/s.

Byron Aldridge (USGS) estimated a flood discharge that ranged from 54,000 to 61,000 ft^3/s by critical-depth calculation for a contracted opening through the bridge. Eyewitness accounts confirm that the opening was unobstructed at the time of the peak discharge. The drop in stage through the contraction was about 19 ft and was documented by high-water marks. The downstream marks were only 3 ft above the after-flood streambed. Aldridge reported that an average velocity of about 75 ft/s would be required to pass the computed discharge through the 160-ft wide bridge opening, thus bringing into question the validity of this contracted-opening estimate. Extensive erosion during and after the peak discharge reduce the reliability of this estimate. Reported deposits of approximately 40,000 yd^3 of new material on the delta at the mouth of Bronco Creek verify that extensive erosion did occur. It is not known if this is an artifact of the peak or of flow duration.

Channel slope in the basin ranges from 400 to 500 ft/mi. Two single cross-section slope-conveyance estimates and an approximation were made to try to confirm peak flow from each of the three subbasins comprising the Bronco Creek drainage. The results of these estimates are 9,100, 18,900, and 10,000 ft^3/s, respectively, for Bronco Wash (one-section slope conveyance), Bronco Creek (one-section slope conveyance), and Greenwood Wash (approximation based on similarity to Bronco Wash). The reported composite estimate is 38,000 ft^3/s.

Carmody (1980) (from House and Pearthree, 1995) used hydroclimatic techniques to estimate runoff in the Bronco Creek basin. He estimated that sustained precipitation of 10 in/hr for 35 minutes would be required to produce a discharge of 73,400 ft^3/s. The only precipitation data seem to be from nearby Wikieup, Ariz., where 3 in. of rainfall was measured in 45 minutes. This translates to about 4 in/hr. Transferring this rainfall data onto the Bronco Creek basin may stretch the technique depending on how widespread the most intense part of the storm was. The eyewitness account of the flood include a 2-hour observation by E. Fancher, Arizona Department of Transportation, of peak or near peak flow, which would support the theory of sustained intense precipitation over the basin.

House and Pearthree (1995) used paleoflood techniques to estimate peak discharge in each of the three subbasins. They combined these results to arrive at a new estimate for the 1971 peak discharge. To avoid the problem of unknown amounts of erosion and (or) fill, they selected bedrock reaches for their study. The study reaches in Bronco Creek, Bronco Wash, and Greenwood Wash were 45, 246, and 87 ft in length, respectively. Step-backwater techniques were used to try to match the slope of what are described as “unequivocal relic high-water marks” and slack-water deposits. The high-water marks are flotsam (woody debris) and were deposited on bedrock shelf-like features. There were from four to six high-water marks found in each reach. The computations were made with a variety of “*n*” values and discharges, and assumptions of subcritical and supercritical flow, until the computed profile matched the high-water mark profile. The cross sections ranged from about 9 ft apart to a maximum of about 30 ft apart. Cross-section widths ranged from about 24 ft for the Bronco Creek reach to about 45 ft for the Bronco Wash reach. The resulting discharge estimates are: Bronco Wash, 8,500 ft^3/s; Greenwood Wash, 3,900 ft^3/s; and Bronco Creek, 14,100 ft^3/s. The total discharge ranged from 26,500 to 30,000 ft^3/s and is considered an upper limit by the investigators.

Hjalmarson and Phillips (1996) used translatory wave theory to estimate discharge for the 1971 Bronco Creek flood in Arizona and the 1974 Eldorado Canyon flood in Nevada. Both basins have steep-gradient, alluvial channels and both have produced extraordinarily high peak discharges. Eyewitness reports of the Bronco Creek flood document a wave extending bank-to-bank about every 4 to 5 minutes. Waves 1,200 to 1,500 ft upstream of the bridge would take from 30 to 45 seconds to reach the bridge. The largest of these waves were 4–5 ft high. There is speculation that these waves deposited the high-water marks used to define the profiles used for the slope-area computation and represented only a transitory peak stage. This phenomenon has been documented in steep, rectangular channels by other investigators (for example, Holmes, 1936), usually where the channel gradient begins to flatten.

Possible sources of error: There are several sources of error in slope-area indirect flow measurements in steep-gradient alluvial channels, particularly for “flashflood” type events. Because of the instability of the bed, it is difficult to know the geometry of the cross sections at the time of the peak discharge or whether the sand was transport like a conveyor belt with minimal net change in cross-sectional area. It is also difficult to estimate what bed forms were present during the peak making assignments of roughness coefficients more difficult than usual.

For the Bronco Creek flood, even the high-water mark data that were used to develop the high-water profile is questionable because of the eyewitness report of bank-to-bank waves. There is speculation that wave crests, reported to be “100 percent” higher than the water surface, deposited the marks. The flow probably contained a high sediment concentration.

Sources of error associated with the contracted-opening (critical-depth) computation at the bridge primarily are due to unknown bed scour during the peak discharge. The bridge does not appear to provide much of a contraction. The hydrometeorologic analysis is burdened with the complexity of no precipitation data in the basin. Errors associated with transporting precipitation intensity data from outside the basin probably are much greater for convection storms than for area-wide storms. There is some evidence from the eyewitness reports of the flood peak at the U.S. Highway 93 bridge that indicates the intense rainfall may have lasted longer than originally thought.

The step-backwater analyses of stable reaches in the three branches of the Bronco Creek drainage were run through reaches as short as 45 ft. Step-backwater analysis does not have much sensitivity when applied to such short reaches of channel; however, for the critical-depth method, this reach length may be sufficient. When analyzing tributary flow, it is always difficult to know if each tributary peaked at the same time. The channel has a steep gradient and is alluvial upstream of the selected reaches so non-Newtonian flow is a possibility, but there is no evidence of a debris flow. The eyewitness account of bank-to-bank waves prompted an analysis of the flood using translatory wave theory. Flow probably was unstable or waves would not have developed. These kinds of flow instabilities need further work because they likely occurred in several of the floods reported herein. This is especially important because guidance in USGS documents about how to handle waves in peak discharge determinations is ambiguous (Benson and Dalrymple, 1967; Rantz, 1982).

Recommendations of what could have been done differently: Given a good reach with good high-water marks, the first approach would still be to make a slope-area measurement. Consultation with the best river mechanics and hydraulics professionals should have been part of the review process as soon as the original computation resulted in an unreasonably high unit discharge. If this team approach had been used, methods such as those of House and Pearthree (1995) might have been done earlier when more and better high-flow evidence was available. The USGS Arizona Water Science Center did a good job of trying to verify peak discharge with flow estimates at the bridge, for individual subbasins, and by using translatory wave theory.

Site visit and review: The site was visited on August 27, 2003, by John Costa (USGS Office of Surface Water), Terry Kenney (USGS Utah Water Science Center), Kenneth Wahl and Gary Gallino (USGS retired), and Kyle House (Nevada Bureau of Mines and Geology). The review team discussed the differences in the peak flow estimates. They agreed that there probably was a base peak flow represented by the House and Pearthree (1995) computations and roughly verified by the individual basin estimates by the USGS. They also agreed that it is difficult to refute the eyewitness account of large waves moving through the slope-area reach. These waves, and the high-water mark evidence, support an estimate of a high instantaneous peak flow with a low volume.

Recommendations: Qualify peak flow as representing two kinds of peak discharges—a base flood peak related to the runoff from the rainfall and a much larger instantaneous peak discharge related to large translatory waves caused by instability in the floodwater.

Eyewitness accounts verify a base floodflow with periodic bank-to-bank waves superposed on the surface of the flow. Estimates of the base (steady) floodflow range from about 28,000 to 38,000 ft^3/s. This peak is associated with the rainfall-runoff and would produce a unit discharge of 1,470–2,000 $(ft^3/s)/mi^2$. The instantaneous peak should be reported as 96,800 ft^3/s as computed by Hjalmarson and Phillips (1997) using wave theory—a discharge that was caused by the steep channel and erodible bed material. Both peaks should be rated as an estimate.

Figure A179. View looking downstream at cross section 1, Bronco Creek near Wikieup, Arizona, August 1971.

Figure A180. View looking upstream toward slope-area reach from U.S. Highway 93 bridge. Bronco Creek near Wikieup, Arizona, August 2003.

Figure A181. View from right bank to left bank near cross section 1, Bronco Creek near Wikieup, Arizona, August 2003.

Figure A182. View toward right bank at cross section 1, Bronco Creek near Wikieup, Arizona, August 2003.

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11067000 Day Creek near Etiwanda, California

(Discontinued gaging station in the Santa Ana River basin, USGS California Water Science Center)

Review of peak discharge for the flood of January 25, 1969

Location: Lat 34°11’06”, long 117°32’20”, in NW 1/4 NW 1/4 SW 1/4 sec.8 T.1 N., R.6 W., San Bernardino County, Hydrologic Unit 18070203, on left bank, 0.5 mi downstream from confluence of two main forks, and 4 mi north of Etiwanda.

Published peak discharge: There is no acceptable published peak discharge for this flood. The stage is published as 9.90 ft, the highest stage measured at the gaging station between 1928 and 1972 when the gaging station was discontinued.

Drainage area: 4.56 mi^2.

Data for storm causing flood: The area of the San Gabriel Mountains in southern California was subjected to intense local storms for more than a week between January 18–26, 1969. A stagnant low-pressure system over the Pacific Ocean sent streaming waves of moisture-laden air into southern California as a succession of storm fronts. Storm total precipitation at Etiwanda was 15.45 in., of which 8.07 in. fell January 24–25, 1969, just prior to the debris flow (Singer and Price, 1971). More than 42 in. of rain fell at high elevations in the San Gabriel Mountains at Lytle Creek Ranger Station, 7.5 mi north-northeast of Etiwanda (Scott, 1971; Singer and Price, 1971). Brush fires the previous year caused extraordinary runoff and numerous debris flows in burned basins. Scott (1971) reports numerous debris flows from small drainage basins in the Glendora area. Day Creek is only 18 mi east of Glendora and in the same geologic setting. Historical photographs taken after the flood of January 25, 1969, and photographs taken during the 2003 review and described herein are provided in figures A183–A190.

Method of peak discharge determination: A four-section slope-area indirect discharge measurement was made on February 8–9, 1969. The reach was selected downstream of the streamflow-gaging station at the head of an alluvial fan extending downstream of the canyon mouth of Day Creek. There were large variations in subreach discharge results (19,300–47,600 ft^3/s) and a significant expansion. Conveyance ratios were exceeded in all reaches. The four-section slope-area result was 29,740 ft^3/s. During review, these comments were made:

> “I don’t believe we should use the results of the four section slope area reach. I don’t believe that the changes in areas of the sections are indicated as changes in slope. If the slope does not decrease with area increase, either a very large increase in “n” takes place or the discharge is increasing between sections. Since we feel quite sure that the above isn’t taking place, there must be either an error in profile or section area. I feel quite strongly about the definition of the cross sections. It may be that high water marks defined both banks at the level indicated but I very much doubt if it did this at the same time. I believe that the flow meandered back and forth as debris blocked the flow. Probably no section completely describes the true flow area but since No. 1 is the smallest, it comes closest. Since slope doesn’t change through the reach (an indication that area has little effect on the flow), I would suggest we use the minimum section and compute Q = KS1/2 and rate the result poor.” (signed L.A. Martens, 3-3-69).

Using the section with the smallest cross-sectional area and nearest the head of the alluvial fan, the slope-conveyance indirect-discharge measurement was calculated to be 9,500 ft^3/s and called an estimate.

Possible sources of error: The most significant error in this indirect discharge measurement was the misinterpretation of this event as a water flood. At and downstream of the streamflow-gaging station, botanical, sedimentological, and geomorphological evidence is unequivocal that at the streamflow-gaging station, the peak flow in January 1969 was a debris flow. Debris flows occurred all around this area from the 1969 storm and were well documented (Scott, 1971). Downstream of the streamflow-gaging station, the middle and downstream parts of the coalescing alluvial fans of Day and Deer Creeks experienced significant flooding (Singer and Price, 1971).

This streamflow-gaging station was operated from 1928 to 1972. The site is extraordinarily difficult to measure high flows because of the volume of sediment moved, an unstable channel, multiple flow paths, steep channel (slope of 0.088), and debris flows that have occurred. The five largest peaks in the period of record were based on indirect methods and determined as:

1. 1938 (five values determined; 4,200–44,000 ft^3/s), final value is based on estimated rainfall-runoff.
2. 1943 (two values determined; 720–1,500 ft^3/s), final value is based on arbitrary estimate.
3. 1950 (six values determined; 580–852 ft^3/s), final value is based on slope-area analysis.

4. 1966 (three values determined; 800–1,740 ft^3/s), final value is based on gage height and field estimate.

5. 1969 (two values determined; 9,450–29,740 ft^3/s), final value is based on slope-conveyance (determined herein to be unreliable).

USGS should not have tried to measure peak discharges at this site. None of the five largest flows in 45 years is based on direct measurements or rating curves. Photographs from the 1938 flow present strong evidence that the peak was a debris flow, not a water flood. None of the five largest peak discharges for this site should be considered reliable, and evidence indicates one and perhaps two were debris flows, not water floods.

Recommendations of what could have been done differently: If this event had been correctly identified as a debris flow, an indirect discharge measurement would not have been attempted. In describing the debris flows from this storm near the Glendora, Calif. area, Scott (1971, p. C247) reported:

> "Inspection of the channels indicates that normal indirect measurement of peak discharge would give extreme values and that sometimes the resultant values probably would exceed enveloping curves developed for maximum floods in small drainage basins in southern California."

Field work should have focused on the factors that are most significant to debris flows, such as failure volume, source materials, and valley geomorphology, and not peak discharge. A very slow-moving debris flow can have a high stage at a gage site but produce a small peak discharge. Field evidence documented below indicates that at the streamflow-gaging station, the debris flow was not moving rapidly.

Site visit and review: To document the interpretation that the January 1969 flow at Day Creek near Etiwanda was a debris flow, several debris-flow experts made a field reconnaissance of the original field site on September 25, 2002. The results of the field trip are reported in a memorandum to K. Michael Nolan, USGS Western Region Surface-Water Specialist, dated September 30, 2002, authored by Thomas C. Pierson and Jon J. Major, research scientists with USGS. Most of their memorandum is quoted here:

> "SUBJECT: Field visit to Day Creek (CA) Indirect Measurement Site, 9/25/02
>
> At your request, we accompanied a field party to visit the site where an indirect measurement of the January 25, 1969, "flood" was made near the USGS stream gage at Day Creek nr Etiwanda (11067000) in the San Gabriel Mountains, just north of Ontario, California. The field party consisted of USGS staff (yourself, Robert Meyer, Dale Cox, Jim Bowers, Bill Kirby, Bob Jarrett, and the two of us) and private consultants (Martin Becker, Doug Hamilton, and Phil Schaller). The purpose of the site visit was to determine, by examination of remaining deposits and other field evidence, whether the "flood" of January 25, 1969, had been a debris flow or a water flow that had transported a large volume of coarse sediment. We understood that no flood or debris-flow events larger than the 1969 event had occurred in this drainage since that time. We also understood that the indirect measurement that was made shortly after the 1969 event by CA district staff had resulted in an unusually high and controversial peak discharge value, which had been entered into the USGS peak-flow database.
>
> We examined the sedimentologic and morphologic characteristics of the deposits along part of the original indirect measurement reach—from the site of the now discontinued gaging station to just upstream of the apex of the fan. This reach, several hundred feet long, is bounded approximately by cross sections 1 and 3 of the original survey. We compared the present-day deposit morphology to that of deposits left by the event in question in the photographs dated February 7, 1969, in order to verify as best we could that we were examining deposits from the 1969 event. We conclude that the 1969 event was definitely a debris flow and not a water flood, at least in the reach we examined. The evidence we found leading us to this conclusion includes:
>
> 1. The remnants of the highest recent deposits in the valley cross section (matching the positions of the 1969 deposits shown in the 1969 photographs) show that the original depositional surface was broadly convex, with lobate lateral and frontal margins.
>
> 2. The lobes of debris were about 1-2 m high and had coarse clasts (boulders) concentrated on the outer margins of the lobes; the bouldery rims held back finer grained debris.
>
> 3. Except at the margins where boulders were concentrated, the deposit exhibited a clast-supported, extremely poorly sorted texture with no visible stratification. All voids between clasts were completely and tightly packed with matrix material.

4. The matrix material was dominantly coarse sand to fine gravel in size but with a few percent of fines (apparently mostly silt); in some places the matrix material was loose and in other places it was slightly cohesive.

5. Coarse clasts on the surface and exposed in cut banks within the main body of the deposit appeared to have random orientations (i.e. no imbrication), although some fabric had developed along the bouldery flow margins.

6. The upstream ends of the stone-masonry side walls of the weir structure that had been constructed at the gage site were not chipped or battered (even the mortar between the blocks).

7. Trees (live oaks) buried in about 1 m of coarse debris from the 1969 event showed no abraisional damage on the upstream sides of their trunks; they were alive and appeared to be quite healthy.

8. The 1969 photos reveal that the bouldery deposits left by the flow filled the weir box to within about 1 m of the bottom of a steel foot bridge mounted on the stone walls above the weir. The upstream sides of the bridge beams showed no evidence of impact by debris. There were no dents and the paint had not been chipped or abraided. This indicates that the bridge had not been touched by the flow.

The physical evidence and characteristics listed in points 1–5 are typical of debris flows and are not found where water floods have transported the debris. Water floods may deposit convex bars but do not leave behind broadly convex deposits, which debris flows typically do. Water floods do not leave lobate deposits, but such lobes are in fact a diagnostic feature of debris-flow deposits. Although floods transporting coarse debris may not leave well-stratified deposits, they usually do leave localized pockets of well-sorted sand and gravel, which we did not see. Except at deposit surfaces where fluvial reworking had probably taken place, the deposits we examined were everywhere extremely poorly sorted. Coarse flood debris typically has numerous small voids between coarse clasts; the coarse clasts in the deposits we examined were all tightly packed with matrix material. Finally, cobbles and small boulders in flood deposits typically show some degree of imbrication (i.e. a type of sediment fabric where a—b axis planes of relative flattened clasts dip upstream); the clasts in the main body of the deposit we examined seemed to be randomly oriented, although fabrics had developed somewhat along the bouldery margins of the deposit (characteristic of debris-flow margins).

The observations described in points 6–8 demonstrate, furthermore, that the event could not have been a water flood. A flood of water generating sufficient shear force to transport the many boulders observed having mean diameters between 0.5 and about 1.0 m would have to have been deep enough to heavily damage or wash away the foot bridge on the weir. In addition, flow velocities would have been high enough to propel cobbles and small boulders into the upstream ends of the weir walls to cause damage there and on the upstream sides of tree trunks in the flow. Because the weir walls and the bridge were completely undamaged and trees were gently surrounded by debris, we infer that the debris-flow surges that spread through this valley reach were moving slowly, probably no more than 1–2 m/s. The slow velocity was probably due to frictional resistance provided by the bouldery surge fronts.

The photos taken on February 7, 1969, show that the fresh January deposit had been eroded very little. The water in the creek was flowing in a narrow channel on the left side of the valley that appears to have been less than 1 m deep. Between that time and today, the deposit experienced more severe erosion by water flow (with relatively little deposition). It is likely that this erosion occurred in late February 1969, when a storm having a rainfall magnitude nearly equal to the storm that triggered the January debris flow occurred but failed to trigger a significant debris flow (according to Robert Meyer).

It is likely that the 1969 debris flow transformed to a more dilute type of flood flow (probably hyperconcentrated flow) at some point farther downstream on the fan. However, disturbance of deposits by the more recent construction of the debris basin at the fan apex and the lack of ground-based photo documentation that would enable identification of 1969 deposits has precluded determination of where the transformation might have taken place. In any case, the published peak-flow value of 9,500 cfs for the January 25, 1969, debris flow is definitely not valid."

Consultants invited to attend this field trip have produced memoranda of their own that argue that the February 1969 event at the streamflow-gaging station was not a debris flow, but a water flood. Most of their memoranda pertain to arguments of the differences to public safety between water floods and debris flows, and other examples of floods and debris flows in other locations, or in areas downstream of the streamflow-gaging station. One memorandum reports results of a sensitivity analysis on the cross-sectional data from the indirect discharge measurement but assumes velocities for the flow that were at odds with field evidence at the streamflow-gaging station (Douglas Hamilton, External Memorandum, November 1, 2002). Another memorandum correctly concludes that both debris flow and water floods are active in Day Creek Canyon during large runoff (Phillip Shaller, Summary of Observations, Field Trip of September 25, 2002). The preponderance of evidence points to the January 25, 1969, event as a debris flow, an event incompatible with the streamflow record at the streamflow-gaging station.

Recommendations: The January 1969 peak discharge is indeterminate, and no meaningful value can be entered into the Peak-Flow File for this event. The stage for this event, 9.90 ft, remains with no discharge associated with it. Individuals interested in the flow history at this site need to exercise due diligence in interpretation of the 1969 debris flow. The record stage but no discharge are clear indications of an unusual event with extensive photographic and written documentation in USGS files.

Figure A183. Unknown location in Day Creek, California, following debris flow showing abrasion on trees, but no major damage in spite of very coarse materials being moved.

Figure A184. View looking upstream toward slope-area reach, Day Creek near Etiwanda, California, February 1969.

Figure A185. View looking downstream at left bank at cross section 2, Day Creek near Etiwanda, California, February 1969.

Figure A186. View looking downstream from cross section 3 toward cross section 4, Day Creek near Etiwanda, California, February 1969.

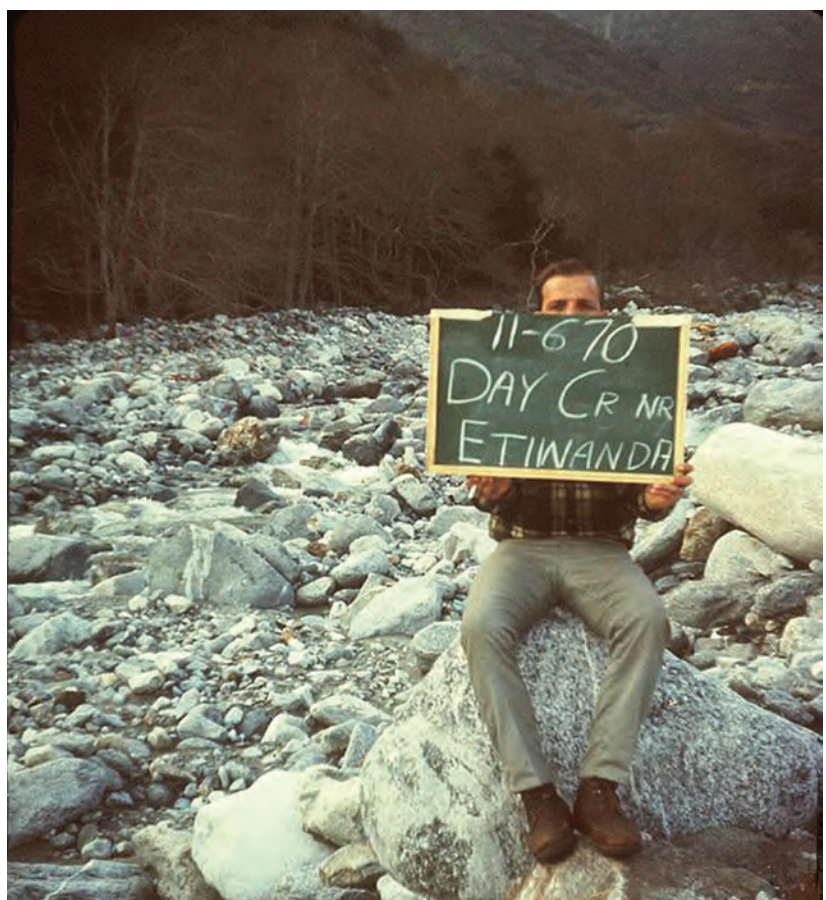

Figure A187. View of slope-area reach, Day Creek near Etiwanda, California, November 1970.

Figure A188. View of debris-flow lobe in slope-area reach, Day Creek near Etiwanda, California, November 1970.

Figure A189. Debris-flow deposit arrested by tree near slope-area reach, Day Creek near Etiwanda, California, September 2002.

Figure A190. Boulder front of debris-flow lobe, Day Creek near Etiwanda, California, September 2002. Flow is from left to right.

11477000 Eel River at Scotia, California

(Gaging station in the Eel River basin, USGS California Water Science Center)

Review of peak discharge for the flood of December 23, 1964

Location: Lat 40°29’30”, long 124°05’55”, in SW 1/4 sec.5, T.1 N., R.1 E., Humboldt County, Hydrologic Unit 18010105, near center of span in left pier of A.S. Murphy Memorial Bridge on State Highway 283, 0.5 mi north of Scotia, and 6 mi upstream from Van Duzen River.

Published peak discharge: The published peak discharge determined by rating-curve extension was 752,000 ft^3/s at a stage of 72.0 ft and should be rated poor.

Drainage area: 3,113 mi^2.

Data for storm causing flood: The 1964–65 flooding was documented by Waananen and others (1971). According to Waananen and others (1971, p. A1)

> “The flooding was caused by three principal storms during the period December 19 to January 31. The December 19-23 storm was the greatest in overall intensity and areal extent. Crests occurred on many major streams December 23, 1964, 9 years to the day after the great flood of December 23, 1955… All the storms, and particularly the warm torrential rain December 21-23, reflected the combined effect of moist unstable air masses, strong west-southwest winds, and mountain ranges oriented nearly at right angles to the flow of air.”

A rating curve, historical photographs taken after the flood of December 23, 1964, and photographs taken during the 2003 review and described herein are provided in figures A191–A196.

Method of peak discharge determination: The peak discharge is based on a rating-curve extension. According to the current (2007) station description in NWIS,

> “Maximum Discharge, 752,000 ft^3/s, Dec. 23, 1964, gage height of 72.0 ft. from floodmarks, from rating curve extended above 220,000 ft^3/s on basis of maximum flow at upstream stations.”

Possible sources of error: During a review of rating curves for this gaging station by the USGS Ukiah Field Office, some measurements made in the 1940s were “left off” the rating developed in 1955 for the 1955 peak discharge. The 1955 peak discharge was 541,000 ft^3/s at a stage of 61.90 ft. The Field Office suggests that including the 1940 measurements would change the 1955 rating and the 1964 extension. The Field Office analysis indicated that a change for the 1964 peak discharge from 752,000 to about 590,000 ft^3/s might be in order.

The reasons for questioning the peak have been summarized by the USGS California Water Science Center as:

1. Measurements 171 and 172 made in February 1940 at 208,000 ft^3/s (stage 44.30 ft) and 304,000 ft^3/s (stage 52.19 ft), respectively, were not used in later ratings. These are the highest and third highest measurements made at the site.

2. The 1955 (and 1964) rating curve was drawn with a slight bend to the right to accommodate the estimated discharges. The California Water Science Center determined there was no overflow at the Scotia gaging station for either of these floods, so the only other factor that would cause the bend to the right would be scour, which would be possible considering the bed composition. However, this would have required about 10 ft of scour. Although the issue has never been debated, there is a possibility that the Van Duzen River, which enters the Eel River about 7–8 mi downstream, caused backwater at the Scotia gaging station.

Recommendations of what could have been done differently: The reasoning behind the decision to not use the 1940 measurements in the 1955 and subsequent ratings should have been documented.

Crews were brought in from outside to aid California personnel in documenting the flooding. There is also ample evidence of using a systems approach to define the various peak discharges; that is, the peak discharges (and the associated daily mean discharges) were compared to other peak discharges in the basin to assure internal consistency of the resulting numbers. The extraordinary flooding also was thoroughly documented in Waananen and others (1971).

Site visit and review: Kenneth Wahl, who was the USGS California District Surface Water Specialist at the time, visited this and other area gaging stations during May 25–27, 1976, specifically to look at the 1964–65 flood levels. A telephone conference between the current review team and representatives of the USGS California Water Science Center took place July 11, 2003, and during the week of July 13, 2003, John Costa (USGS Office of Surface Water) visited the gaging station and reach.

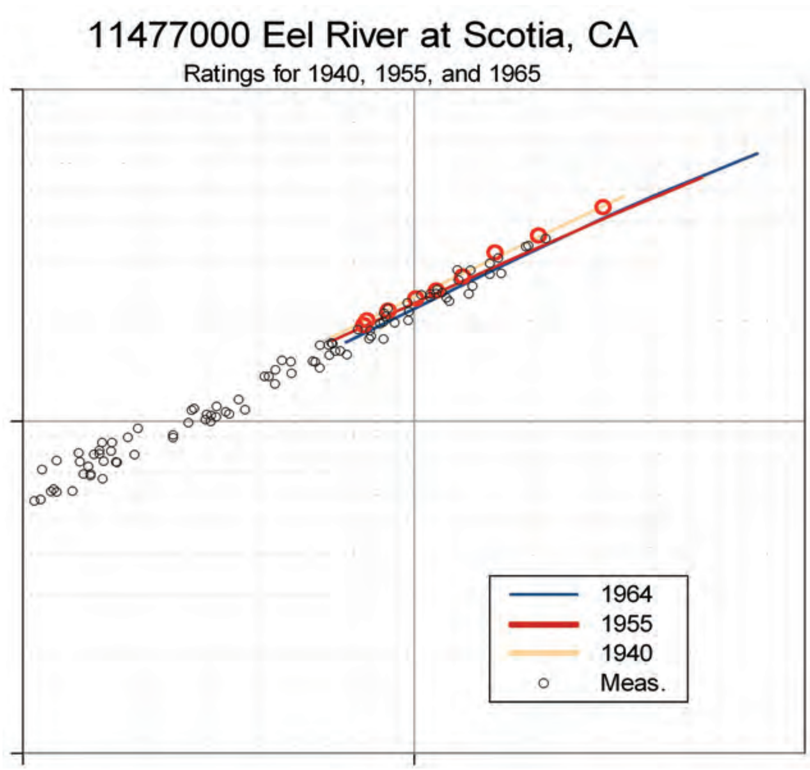

Figure A191. Rating curves for Eel River at Scotia, California in 1940, 1955, and 1964 with pre-1950 measurements indicated in red. Note that including the highest and third highest measurements from 1940 would pull the 1955 and 1964 ratings to the left.

A review of all high-water measurements shows that 1940 measurements 171 and 172 are indeed the highest and third highest measurements available. However, measurements made in 1953, 1956, and 1963 in the range of 193,000 to 217,000 ft^3/s show stages about 2 ft below those that would be expected from the 1940 measurements. Measurement 173 (161,000 ft^3/s) made in 1940 has a stage 1.25 ft higher than measurement 360 (164,000 ft^3/s) made in 1955. In fact, the 1940 measurements, for whatever reason, define the left-most measurements in the cloud of all high measurements. Stage versus width and stage versus velocity plots show that the 1940 measurements consistently define a slightly different relation than most of the other high measurements. However, nearly all the more recent high measurements were obtained with optical current meters, and these data were converted to mean velocity.

This suggests that the decision to not include those measurements in defining the 1955 rating was not an oversight but was based on comparisons of the data. Speculation by the review team is that the decision was made as part of the nationwide 1950 compilation review (U.S. Geological Survey Water-Supply Papers 1301-1319, published between 1954-61) in which all past ratings for an individual station were overlaid on a single plot.

Just downstream of the gaging station, the river did overflow the left bank, although that overflow was not extensive. A plot of measurement width versus stage shows a decided increase in width for stages above about 30 ft, and width for the highest measurements is about 800 ft. The elevation of the 1964 flood was about 107.5 ft above mean sea level (72.0 ft stage + 35.5 ft datum). Superimposing that elevation on the topographic map shows the flood width of about 1,800 ft at the bend downstream; the channel width at that point is about 1,000 ft. About 2 river miles downstream, the river exits the Scotia Bluffs; at this point, the right bank overflowed extensively (Waananen and others,1971, fig. 20, p. A66). The slope of the channel through this reach is about 5 ft in 10,000 ft (0.0005 ft/ft). With a depth of more than 60 ft and a slope of only about 2.6 ft/mi, overflow several miles away can have an effect on rating shape.

On July 10, 2003, Kenneth Wahl spoke to Rio Dell City Councilman Bud Leonard and Karen Hall, an employee of the Rio Dell City Hall. Both commented that the city lost property that was stored near the sewage treatment plant. Ms. Hall who lived just downstream and a little higher than the plant said her house was not affected. She did say, however, that houses downstream of the plant were flooded.

Recommendation: The original peak discharge of 752,000 ft^3/s should be accepted as published.

There is significant uncertainty in the 1964 peak discharge because (1) it is based on extending a rating curve that did not include the first and third largest measured floods in the gage history, and (2) most of the recent highest flows were measured with optical meters and converted to mean velocity. If this evaluation were done in 1955, the argument to base the rating extension for 1955 and 1965 on the highest measurement (made 1940) might be more compelling. However, in 2003, hydrologists have the benefit of all data collected since 1955. There are now 27 measurements of 100,000 ft^3/s or more. The 1940 measurements define the left envelope for a composite rating. Given all the data available, the band of reasonable extensions would range from about 600,000 to about 800,000 ft^3/s. The lower values would place more emphasis on the 1940s measurements; the higher values would place emphasis on the overflow.

Figure A192. View from right-bank flood plain to left bank following flood in 1964, Eel River at Scotia, California. Streamflow-gaging station located on this bridge.

Figure A193. View of downstream bridge from streamflow-gaging station, Eel River at Scotia, California, during flow of about 200,000 cubic feet per second on February 17, 2004.

Figure A194. View of downstream bridge from streamflow-gaging station during low-flow period, Eel River at Scotia, California, July 13, 2003.

Figure A195. View upstream from streamflow-gaging station during flow greater than 150,000 cubic feet per second, Eel River at Scotia, California, December 15, 2002.

Figure A196. View looking upstream from streamflow-gaging station during low-flow period, Eel River at Scotia, California, July 13, 2003.

Little Pinto Creek Tributary near Newcastle, Utah

(Miscellaneous ungaged site in the Virgin River basin, USGS Utah Water Science Center)

Review of peak discharge for the flood of August 11, 1964

Location: This flood was located about 11 mi northwest of New Harmony, Utah, at 37.5894N and 113.4486W.

Published peak discharge: The published peak discharge for this flood is 2,630 ft^3/s and is rated poor.

Drainage area: The drainage area of 0.30 mi^2 was determined by planimeter from the Page Ranch quadrangle map, scale 1:24,500. The drainage-area computations are included with the indirect measurement.

Data for storm causing flood: The measurement summary includes the following sentence:

> “Cloudburst storm of unusual intensity which caused heavy runoff on several streams in the Pine Valley Mountains.”

Little else is known about the storm. There is no evidence of similar flooding on any area gaging stations, but there are few gages in the vicinity and none on small streams. A photograph taken during the 2003 review and described herein is provided in figure A197.

Method of peak flow determination: Discharge was determined by a two-section slope-area method. The measurement survey was conducted October 15, 1964. The computation was straightforward. Only eight high-water marks were obtained on each bank, but the resulting profiles were well defined and had more than 6 ft of fall in the 73-ft reach between the two sections (s = 0.082 ft/ft); there was fair agreement between the banks with the left bank showing about 0.5 ft of superelevation through the reach. The computation treated both cross sections as unit sections (no subdivision) and used n=0.045. The field notes showed subdivision of both banks on the basis of shape but suggested an n-value increase to 0.050 only for the left bank. Elmer Butler, who ran the rod for the survey, reviewed the measurement and did not recommend the subdivision; the authors agree with Butler that the subdivision is not necessary.

The computations and summary were done by J.K. Reid, checked by “L.S.,” and reviewed by Elmer Butler. Because Butler ran the rod on the survey, the measurement had no independent outside review. Butler’s review note written on the measurement summary notes that,

> “This flood represents the highest known unit rate of runoff in the State (8,770 cfs/sq mi).”

As part of this 2003 review, the original computation was run through the current slope-area computation (SAC) program. The resulting discharge of 2,640 ft^3/s confirms the original computation of 2,630 ft^3/s. The reach contracts by about 6 percent, and about 90 percent of the total energy loss was due to friction loss. Mean velocities were 18 ft/s, mean depths are less than 3 ft, and Froude numbers are 1.84 and 1.99. These values of Froude number are high.

Possible sources of error: The most obvious source of uncertainty is in the roughness values associated with slopes of about 8 percent. However, the values used seem to be consistent with verification data collected for more moderate slopes. The other principal source of uncertainty is in the drainage area. Even a small change in the basin boundary would have a significant effect on the final drainage area. That drainage area, however, was determined from a 1:24,000-scale map that is still the best available.

Recommendations of what could have been done differently: Neither the field notes nor the measurement summary mention why the survey was limited to the approximately 130-ft reach. A longer reach with three sections could add confidence in the final result. The site appears that it could have supported at least one more cross section if the profile had been extended about 50 ft.

Apparently no photographs were taken in 1964 or they were misplaced. Photographs are not optional; they are indispensable in reviewing indirect measurements and in locating the reach and cross sections during later site visits. On the basis of the amount of sand now present on the streambed and highly turbulent, supercritical flow, the n values used may actually be low. However, without photographs, one could only assume from the written summary description of the cross sections that less sand was present in 1964.

Site visit and review: A field visit was made August 26, 2003, by John Costa (USGS Office of Surface Water), Gary Gallino (USGS retired), Dale Wilberg and Terry Kenne (USGS, Utah Water Science Center), and Kenneth Wahl (USGS retired). Because the site was located about 0.75 mi from the nearest road and no photographs were taken in1964, there is no assurance that the reviewers located the exact reach of the original survey. On the basis of the fall, cross-section dimensions, and GPS location, however, it is believed that the appropriate reach was found.

There is considerable evidence of old debris flows along the channel upstream of the survey reach, but the surveyed reach showed none of the characteristics of debris flow. Therefore, the reviewers concluded that the flow had been a water flood. Although the reach appears to be straight on the plan view, the site visit revealed that there is a slight curvature to the right throughout the reach. That curvature, combined with the high velocities, could easily produce the 0.5 ft of superelevation shown for the left bank.

Recommendation: The original peak discharge of 2,630 ft^3/s should be accepted as published.

The flood appears to have been a water flood, the computation was done correctly, and there is no new evidence to support a recomputation.

Figure A197. View looking downstream through slope-area reach, Little Pinto Creek tributary near Newcastle, Utah, August 26, 2003.

Boney Branch at Rock Port, Missouri

(Miscellaneous ungaged site in the Boney Branch basin, USGS Missouri Water Science Center)

Review of peak discharge for the flood of July 18, 1965

Location: This flood was located in the western city limits of Rock Port, Missouri, at 40.4139N and 95.5167W.

Published peak discharge: The published peak discharge for this flood is 5,080 ft^3/s at a miscellaneous site 0.3 mi from the confluence with Rock Creek. The computation was rated fair.

Drainage area: The drainage area of 0.76 mi^2 was estimated from a 1:62,500-scale topographic map. A GIS re-run from a 1:24,000-scale topographic map produced an area of 0.71 mi^2. The basin drains the loess hills west of the town of Rock Port. There are numerous small dams in drainages to create stock-watering ponds and two larger ponds probably designed as detention ponds to slow runoff from storms. It is not known how many of these small ponds were in place at the time of the 1965 flood, but the largest was built afterwards.

Data for storm causing flood: Torrential rainfall covered the entire Boney Branch basin on July 18, 1965. Estimates range from 11 to 18 in. of rain from the storm on July 18 and about 8 in. from a second storm on July 19 (Atchison County Mail, July 20, 1965, edition). A local resident measured 14 in. of rain from the first storm, which caused the peak flow in Boney Branch, and 8 in. from the second storm the next day. Boney Branch is an east-flowing drainage from the loess hills bordering the Missouri River flood plain and is a tributary to Rock Creek that flows into the Missouri River. Rock Creek also flooded and damaged bridges and businesses in downtown Rock Port. Historical photographs taken after the flood of July 18, 1965, and photographs taken during the 2003 review and described herein are provided in figures A198–A211.

Method of peak-discharge determination: Peak discharge was computed from a three-section slope-area measurement. The 400-ft-long reach is slightly curving with a fairly large overflow area along the left bank. The main channel is covered with small brush and scattered trees. The left-bank overflow is grass and is kept short by local residents. The right bank is steep and did not overtop except in the upstream part of the reach. The main channel is very sinuous upstream of the reach. There is a road embankment about 0.25 mi upstream of the reach, but it is unknown if flow was impounded behind this embankment, if the road was overtopped and the embankment failed, or if the road embankment was in place at the time of the peak discharge.

The site was first visited on August 11, 1965. The reach was selected, and a few high-water marks were flagged. These were mostly seed lines on trees near the main channel. The profile and cross sections were surveyed on August 24–25, 1965. The high-water profile is defined by six high-water marks on each bank, most of which were flagged marks selected during the initial site visit. The profiles define a 1.8-ft fall through the approximately 300 ft (slope = 0.006 ft/ft) between sections 1 and 3. The slope of the high-water profile on both banks is essentially parallel. Most of the right bank slumped during or after the peak eliminating most usable high-water marks. The left bank was essentially a lawn maintained by local residents, so most high-water marks along that bank probably were destroyed during cleanup.

Manning's "*n*" values were estimated during the initial site visit on August 11, and the survey party chief concurred with the assigned values. A composite "*n*" value of 0.055 was used throughout the reach even though each of the three sections was subdivided. The left-bank grassy overflow had a relatively low-flow resistance. Roughness in the upstream part of the reach appears to be greater because of a brushy left-bank overflow area just upstream of the upstream section and the sharp curvature just upstream of section 1. The main channel is cut into the loess, so the main flow resistance comes from vegetation and irregular banks.

Computed velocities ranged from about 5.5 to 7.0 ft/s. Froude numbers seem reasonable and are about 0.5 for all three sections. There must not have been much floating debris because a small pipeline crossing the channel was overtopped by about 8 ft of water and was not damaged.

Possible sources of error: The profiles are defined by too few high-water marks. Many of the defining marks are seed lines on trees in the main channel that could have been affected by run-up from surface velocities approaching 10 ft/s. These marks should have been verified by leveling to high-water marks along the flow margins. If this had been done, more high-water marks might have been found along the margins.

A composite "*n*" value probably should not have been used for subdivided sections. Independent estimate of flow resistance should have been assigned for each subdivided area and would have yielded a more defensible result. As an example, approximately 40 percent of the area of section 2 was grass inundated by about 8 ft of water. The same is true for about

25 percent of section 3 and a smaller percentage of section 1. Manning's "*n*" for the main channel would have to be higher (about 0.075) to account for the composite roughness coefficient used for the computation.

It is not known how much rain fell in the upstream part of the basin during the storm. There is no evidence that anyone investigated the possibility of failed storage ponds or ponding upstream of the road embankment.

Recommendations of what could have been done differently: Visiting flood sites as soon as possible after a flood, particularly in developed areas, would improve the accuracy and reliability of data collected. For example, cleanup often starts almost immediately after a flood, and good quality high-water marks can be destroyed. Always verify high-water marks obtained in mid-channel with evidence at the flow margin if possible to eliminate artificially elevated stage caused by run-up on the flow obstructions. Flow-resistance coefficients for each subarea need to be estimated for subdivided sections. Always conduct a reconnaissance of the upstream part of the basin, particularly in small basins, to search for evidence of landslides, erosion, or failed structures that could have a major effect on peak flow. Substantiate results for extreme floods by estimating peak flow in other affected drainages in the area to verify basin yield and spatial distribution of the storm and flooding (Jarrett, 1990).

Site visit and review: The site was visited on May 6, 2003, by John Costa (USGS Office of Surface Water), Rodney Southard (USGS Missouri Water Science Center), and Gary Gallino (USGS, retired). The field-review team interviewed a local resident who observed the flood and collected precipitation data for the storm. The team also visited the local newspaper office and reviewed articles about the storm and flood. Rodney Southard used the WSPRO step-backwater model to analyze water-surface elevations in the measurement reach because of concern about how representative the superelevated high-water marks caused by velocity-head run-up on tree trunks located near mid-channel were of actual water-surface elevations. Model results verified water-surface elevations at the cross sections within acceptable limits. Southard also re-ran the discharge computation using estimated Manning's "*n*" values for each subdivided area. The results were not significantly different than the original computation.

Recommendations: The original peak discharge of 5,080 ft^3/s should be accepted as published and the rating should be assigned as "fair."

The amount and intensity of the storm rainfall make this result reasonable.

Figure 198. View looking downstream of left bank of cross section 3, Boney Branch at Rockport, Missouri, August 1965.

Figure A199. View looking downstream of right endpoint of cross section 2, Boney Branch at Rockport, Missouri, August 1965.

Figure A200. View looking downstream of right endpoint of cross section 3, Boney Branch at Rockport, Missouri, August 1965.

Figure A201. View looking downstream of right endpoint of cross section 1, Boney Branch at Rockport, Missouri, August 1965.

Figure A202. View looking downstream at cross section 1, Boney Branch at Rockport, Missouri, August 1965.

Figure A203. View looking downstream of right bank between sections 2 and 1, Boney Branch at Rockport, Missouri, August 1965.

Figure A204. View looking downstream of cross section 2, Boney Branch at Rockport, Missouri, August 1965.

Figure A205. View looking downstream of cross section 1, Boney Branch at Rockport, Missouri, August 1965. Rod between sections 1 and 2.

Figure A206. View looking upstream of right bank between cross sections 2 and 3, Boney Branch at Rockport, Missouri, August 1965.

Figure A207. View looking upstream of cross section 1 to section 2, Boney Branch at Rockport, Missouri, August 1965.

Figure A208. View looking downstream between cross sections 2 and 3, Boney Branch at Rockport, Missouri, May 6, 2003.

Figure A209. View looking upstream at cross section 2, Boney Branch at Rockport, Missouri, May 6, 2003.

Figure A210. View looking upstream to cross section 2, Boney Branch at Rockport, Missouri, May 6, 2003.

Figure A211. Small agricultural dams in loess headwaters of Boney Branch at Rockport, Missouri May 6, 2003.

Stratton Creek near Washta, Iowa

(Miscellaneous ungaged site in the Stratton Creek basin, USGS Iowa Water Science Center)

Review of peak discharge for the flood of August 9, 1961

Location: This flood was located about 3.8 mi east of Washta, Iowa, at 42.5807N and 95.6417W.

Published peak discharge: The published peak discharge for this flood is 11,000 ft^3/s. The original two-section slope-area result was rated fair but was downgraded to estimate after review.

Drainage area: The drainage area of 1.9 mi^2 drains mostly farmland that was growing crops during the storm.

Data for storm causing flood: The Stratton Creek basin, and approximately 18-20 mi^2 surrounding it, was hit by double-digit precipitation over a period of about 6 hours on August 8, 1961. Precipitation data published by the Iowa Natural Resources Council lists the storm as lasting from about 6 p.m. to 12 p.m. According to local residents most of the rainfall in the Stratton Creek basin fell in about 3 hours and totaled nearly 12 in. Mr. Peterson, a local farmer, measured 12 in. of rainfall in a newly installed stock-watering tank. The tank was dry and level before the storm started. Soil in the upstream part of the basin is rich in clay and has low infiltration rates, so rainfall of this intensity had a high percentage of runoff. Historical photographs taken after the flood of August 9, 1961, and photographs taken during the 2003 review and described herein are provided in figures A212–A231.

Method of peak discharge determination: A two-section slope-area measurement was made for a 500-ft long reach located immediately upstream of a county road crossing about 2 mi east of Washta, Iowa. The high-water profile is defined by a few high-water marks clustered at the ends of the sections. These marks were flagged on the afternoon of August 9. Intense rain continued after passage of the flood peak and made finding reliable high-water marks difficult. The right-bank overflow area was planted in soybeans. The high-water marks in this area were mostly mud or dirt lines on the bean plant leaves. The left bank was mostly sloughed off at the downstream section so no high-water marks were available. The left-bank overflow area at cross section 1 was mostly pasture. The high-water marks found in this area could be superelevated due to the bend in the channel reach. There is a small left-bank tributary that is crossed by section 1. Section 1 probably should have been located about 150 ft downstream to avoid the radical difference in cross-section geometry for the two sections, or a third section should have been surveyed between sections 1 and 2. There was sufficient fall, but the reach had poor high-water-mark definition. The result of the two-section slope-area measurement was 13,300 ft^3/s.

The measurement was closely reviewed because of the high unit discharge. Reviewers at USGS Headquarters in Washington, D.C., suggested verification with a flow-through bridge and flow-over-road computations using section 2 as the approach section. Section 2 is badly skewed to the road section and culvert. There were no high-water marks found downstream, so the road overflow measurement was computed assuming critical depth at the road section. The critical-depth method should provide a reliable peak discharge. This computation resulted in a discharge of about 6,600 ft^3/s over the road and 3,400 ft^3/s through the bridge for a total discharge of 10,000 ft^3/s. This value was combined with the slope-area computation (13,300 ft^3/s), and the final discharge was published as 11,000 ft^3/s.

The Manning's "*n*" values used were in the range of 0.040 to 0.055. Flow depths of 5 to 7 ft over the bean crop and pasture in the overflow area make the roughness coefficients for these areas seem high. The roughness values for the main channel are reasonable considering the 15-ft flow depth and type and amount of vegetation.

Possible sources of error: The base line for the slope-area reach is misaligned compared to the actual flow path. Realignment would reduce the distance between cross sections by about 5 percent, or about 440 ft. This change would not significantly decrease the flow.

The upstream section is a substantially different shape than the downstream section. The conveyance does not change uniformly between the two sections. Cross section 1 should have been relocated, or a third section should have been added at the change-in cross-section geometry.

The high-water profile is poorly defined except at the road crossing. An attempt should have been made to locate high-water marks in the long, fairly straight reach downstream of the county road. This is a good reach to use a step-backwater computation to check the high-water marks at the road embankment. Photographs taken following the flood show a tree lodged near the bridge opening, which could have affected flow through the bridge. There is no way of knowing if the tree was in place at the time of the peak discharge. The road embankment was submerged under 5 ft of water at the time of the peak discharge.

The high-water marks at the right end of cross section 2 could have been affected by water flowing out of the upstream road ditch and over the road. Both ditches probably were flowing full down the steep road grade.

Recommendations of what could have been done differently: A step-backwater model could have been used to compute discharge for the long, straight reach downstream of the bridge and road embankment. When faced with a questionable slope-area reach and a poorly defined high-water profile, it may be better to use the step-backwater model to estimate peak discharge.

A third section could have been surveyed between cross sections 1 and 2 at the substantial change in cross-section geometry. Field personnel should not hesitate to survey an extra section even though the water-surface profile is poorly defined. Be diligent in looking for high-water marks in the best reach available. In this case, the best reach appears to be downstream of the reach that was used.

Site visit and review: The site was visited on May 5, 2003, by John Costa (USGS Office of Surface Water), Ed Fischer (USGS Iowa Water Science Center), and Gary Gallino (USGS, retired). The field-review team toured the basin with Mr. Peterson, a local farmer who lived in the area during the flood. He pointed out locations of damage and of bucket precipitation measurement in a stock tank. As a result of the field review, the USGS Iowa Water Science Center surveyed cross sections and used the HEC-RAS and WSPRO step-backwater models to determine the discharge necessary to match the flow width at the inundated road embankment. The resulting discharge estimates were 11,600 and 9,500 ft^3/s, respectively. These step-backwater model analyses seem to verify critical depth at the road section.

Recommendations: The original peak discharge of 11,000 ft^3/s should be accepted as published and the rating should be rated as “estimate.”

Results of the step-backwater models bracket this discharge. There is too much speculation in some of the data used in the models to recommend change in peak discharge after a time lapse of more than 40 years.

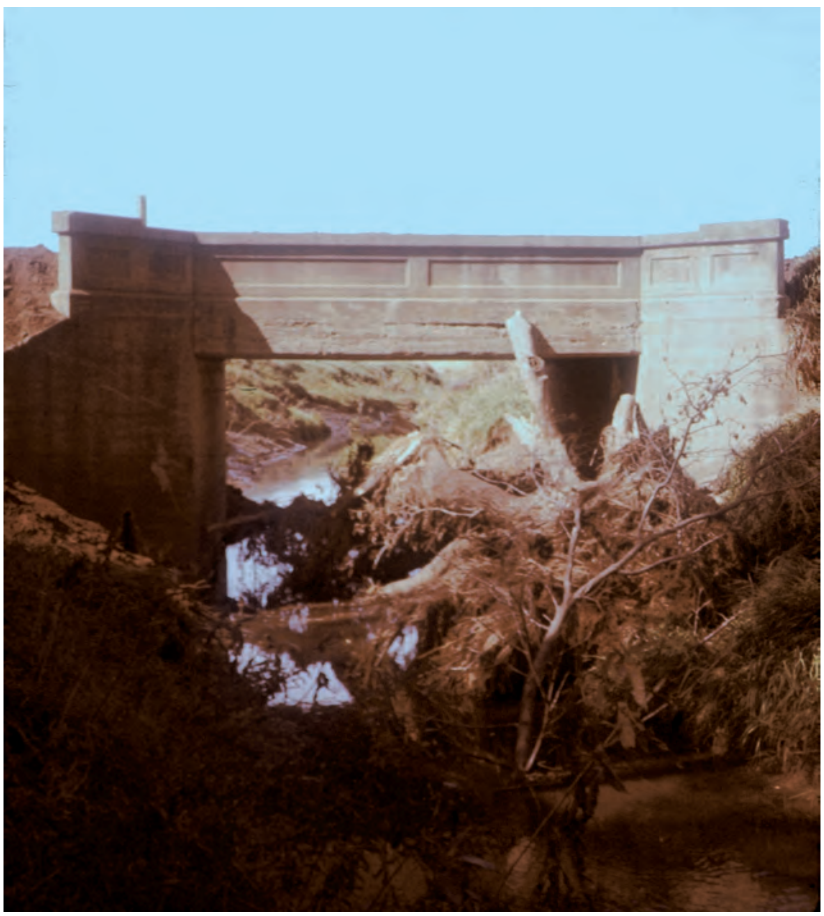

Figure A212. View looking downstream of road crossing and culvert, Stratton Creek near Washta, Iowa, August 1961.

Figure A213. View of right-bank high-water mark at bridge crossing, Stratton Creek near Washta, Iowa, August 1961. Flow is from right to left.

Figure A214. View looking downstream along channel upstream of cross section 1, Stratton Creek near Washta, Iowa, August 1961.

Figure A215. View looking from left bank to right bank long cross section 1, Stratton Creek near Washta, Iowa, August 1961.

Figure A216. View looking downstream of left bank, Stratton Creek near Washta, Iowa, August 1961.

Figure A217. View looking downstream of right bank, Stratton Creek near Washta, Iowa, August 1961.

Figure A218. View of left bank high-water mark at bridge crossing, Stratton Creek near Washta, Iowa, August 1961. Flow is from left to right.

Figure A219. View of right bank high-water mark of downstream side of road and culvert crossing, Stratton Creek near Washta, Iowa, August 1961.

Figure A220. View of slope-area reach upstream of culvert, Stratton Creek near Washta, Iowa, August 1961. Flow is from left to right.

Figure A221. View looking from right to left along cross section 1, Stratton Creek near Washta, Iowa, August 1961.

Figure A222. View looking from right to left along cross section 2, Stratton Creek near Washta, Iowa, August 1961.

Figure A223. View looking toward left bank from downstream side of road and culvert, Stratton Creek near Washta, Iowa, August 1961.

Figure A224. View looking toward left bank from end of cross section 2, Stratton Creek near Washta, Iowa, August 1961.

Figure A225. View looking upstream of culvert and road crossing, Stratton Creek near Washta, Iowa, August 1961.

Figure A226. View looking from right to left bank along road crossing, Stratton Creek near Washta, Iowa, May 5, 2003.

Figure A227. View looking downstream of culvert and road crossing, Stratton Creek near Washta, Iowa, May 5, 2003.

Figure A228. View looking from right to left bank with people standing on approximate high-water mark, Stratton Creek near Washta, Iowa, May 5, 2003.

Figure A229. View looking upstream of culvert and road crossing, Stratton Creek near Washta, Iowa, May 5, 2003.

Figure A230. View looking from right to left bank at culvert crossing, Stratton Creek near Washta, Iowa, May 5, 2003. Flow is from left to right.

Figure A231. View of headwaters of Stratton Creek near Washta, Iowa, May 5, 2003.

Castle Creek Tributary No. 2 near Rochford, South Dakota

(Miscellaneous ungaged site in the Cheyenne River basin, USGS South Dakota Water Science Center)

Review of peak discharge for the flood of July 28, 1955

Location: This flood was located about 5 mi southwest of Rochford, S.D., at 44.0656N and 103.7994W.

Published peak discharge: The peak discharge occurred on July 28, 1955, and was determined from an indirect measurement to be 98.9 ft^3/s (Wells, 1962). During review in 1955, the culvert computation was increased by 1.2 ft^3/s, but the review recommended no revision. The combination measurement (culvert plus road overflow) was rated fair.

Drainage area: 0.0192 mi^2 (about 12 acres). Because of the small area, the perimeter of the basin was defined by transit/stadia survey at the time the flood was surveyed, and the area was determined by planimeter.

Data for storm causing flood: The flooding in the Rochford area was documented by Wells (1962, p. 110-113). According to Wells (1962, p. 110), as much as 5 in. of rain fell in 2 hours in the storm center 6.5 mi southwest of Rochford. Wells (1962) developed an isohyetal map of the area for July 28 and reported results for indirect measurements at eight miscellaneous sites. The July 29, 1955, edition of the Rapid City Journal featured several stories about the storm and resulting floods. A photograph taken during the 2003 review and described herein is provided in figure A232.

Method of peak discharge determination: The peak discharge is based on the sum of discharges through an 18-in. diameter corrugated-pipe culvert (originally 14.2 ft^3/s) and flow over a county road (84.7 ft^3/s). The computations were reviewed in the USGS Central Region by Howard Matthai (July 6, 1956) and at USGS Headquarters by M.A. Benson (July 18, 1956). During the latter review, the original culvert computation (type IV flow) of 14.2 ft^3/s was recomputed as type VI flow to be 15.4 ft^3/s. However, the review suggested that no revision was needed given that the culvert computation was a small part of the total peak discharge (about 15 percent) and the change itself was minor (about 1 percent).

The original measurement summary noted that the culvert entrance condition was unusual and did not exactly fit standard conditions, and Howard Matthai (USGS) concurred with this assessment. Given that the culvert flow was a small part of the total flow, the culvert computation was deemed acceptable.

The water-surface profiles, and more importantly, the fall over the road embankment were well defined. Those profiles show that the roadway clearly acted as a broad-crested weir (or flow-over-road) with good get-away conditions.

Possible sources of error: The most likely sources of error in the measurement are (1) the assumption that the culvert is free from debris and obstructions at the peak, (2) the determination of the culvert flow type and coefficient, and (3) the assumption that the road embankment acted as a broad-crested weir (critical depth occurred). Normally on a basin of this small size, the size of the basin that produced the flood would be questioned, but that question was removed in 1955 by surveying the perimeter of the basin.

Recommendations of what could have been done differently: This is an excellent example of how large floods should be handled. Multiple measurements were made, which give corroborating evidence of the unusual nature of the flooding. The survey of the basin perimeter removed the normal uncertainty that surrounds drainage-area determination for such small areas. Finally, the measurement received critical review at all levels. Apparent loss of the pictures was unfortunate, but only those for this basin were lost; the photographs for the other seven indirect measurements apparently are available.

Site visit and review: Kenneth Wahl (USGS retired) and R.W. Teller (USGS South Dakota Water Science Center) visited the site on May 29, 2003. The original pictures of the site were misplaced in 1955, but the field-note sketches are fairly definitive.

The original 18-in. culvert has been replaced with a 24-in. diameter culvert with spiraled corrugation. Therefore, the unusual entrance condition noted in the survey could not be examined. The county road also has been raised perhaps 1–2 ft and widened; the present culvert is about 50 ft long, whereas the original culvert length was 24 ft. Ralph Teller spoke to the county road crew chief, Heine Junge, who confirmed that the roadway has changed since 1955. It is unlikely, however, that there have been any significant changes to the basin that produced the flood. That basin, which is about 600 ft wide and extends about 1,000 ft upstream, has grass cover and no trees. Upstream from the roadway, the waterways are grassy swales, and there are no defined channels. Flow during extreme storms would essentially arrive at the roadway as sheet flow that converges at the culvert.

Debris blocking the culvert entrance is highly unlikely given the total absence of any sources of woody debris in the basin. That the culvert flow type was either IV or VI seems indisputable given the profiles that were based on the high-water marks and the field notes made at the time of the survey. Either flow type results in about 15 ft^3/s through the culvert. Because of the unusual entrance condition, the culvert coefficient is subject to debate, and the questions were raised in 1955. However, any reasonable reinterpretation of the coefficient would change the culvert discharge by only 2–3 ft^3/s, thus changing the total flow by only 2–3 percent. Finally, the field notes and profiles leave little doubt that the roadway acted as a broad-crested weir and that get-away conditions were such that there was no submergence. Kenneth Wahl (USGS retired) checked the original culvert computations and the computations for road overflow and concluded everything was in order. However, today's TWRI on flow over embankments would give C values for the roadway that are about 7 percent greater than those used in 1955.

Recommendations: The original peak discharge of 98.9 ft^3/s should be accepted as published, but rounded to 100 ft^3/s, and rated fair.

There is no doubt that the rain and runoff from this small basin and from the surrounding area were exceptional. The computations are done correctly, and there is little chance that this was something other than a water flood.

Figure A232. View looking upstream at the basin that produced the 1955 flood, Castle Creek Tributary No. 2 near Rochford, South Dakota, May 29, 2002. The basin perimeter is the grassed ridge in the near foreground (perhaps 300 yards away). The 1955 18-in. culvert has been replaced with a 24-in. culvert, and the road fill has been raised and is about twice as wide.

Wenatchee River Tributary near Monitor, Washington

(Miscellaneous ungaged site in the Columbia River basin,
USGS Washington Water Science Center)

Review of peak discharge for the flood of August 25, 1956

Location: This flood was located about 5.9 mi northwest of Wenatchee, Wash., at 47.4772N and 120.4081W.

Published peak discharge: The published discharge for this flood is 903 ft^3/s and is rated fair. A second slope-area measurement of the same storm was made in a nearby basin (Wenatchee River Tributary No. 2; State of Washington, 1964) and produced a discharge of 1,950 ft^3/s. The unit discharges for these two basins are 6,020 and 1,480 $(ft^3/s)/mi^2$, respectively. The two drainages are only a few miles apart.

Drainage area: The drainage area is 0.15 mi^2 and was planimetered from the 1:24,000-scale quadrangle map for Monitor, Wash. The basin is a small, very steep canyon that heads in a maze of smaller drainages emanating from a fairly flat mesa. The exact basin limits are difficult to define at this scale.

Data for storm causing flood: Monitor, Wash., is located about 10 mi northeast of Wenatchee. A localized, intense rainstorm hit the area at about 5 p.m. on August 25, 1956. The storm is reported to have dropped 2.5 in. of rain on the basin. No timeframe is reported, but the duration probably could be retrieved from the local newspaper or from weather records. Several small drainage basins experienced severe flooding. Flow estimates were made for two miscellaneous sites in the Monitor area—this one and Wenatchee River Tributary No. 2. These small, steep canyons empty onto the Wenatchee River flood plain, and the flood damaged fruit orchards that dominate the sandy valley floor. The smaller of these two basins (Wenatchee River tributary near Monitor, Wash., produced record-breaking unit discharge. According to local residents, this was the worst flood in at least 66 years. Historical photographs taken after the August 25, 1956, flood and during the 2003 review and described herein are provided in figures A233-A239.

Method of peak discharge determination: Two slope-area measurements were made in this basin in two short reaches near the canyon mouth. The reaches were selected about 3 weeks after the storm. The two reaches are separated horizontally by about 200 ft, a stretch where no high-water marks could be found. The slope areas were surveyed to different arbitrary datums and were not referenced to each other. The upstream reach is at the mouth of the canyon and extends upstream about 74 ft. The downstream reach is in an area where the bottom of the channel had been filled to provide an area to plant fruit trees. This reach is about 70 ft long. Three cross sections were surveyed in the upstream reach, and two cross sections were surveyed in the downstream reach. Only sections A and B were used in the upstream reach because of 37-percent expansion from section B to section C. The resulting discharges are:

Upstream reach sections A and B	1,010 ft^3/s
Downstream	796 ft^3/s

Both reaches were considered poor even though the high-water profiles were defined by good to excellent high-water marks. It was decided that the best result could be obtained by averaging the two discharges.

The upstream portion of the basin is extremely steep. The elevation change from the slope-area reach to the top of the drainage is more than 1,150 ft. The drainage is only about 1 mi long; thus, the average slope is about 22 percent. There were cloud-seeding operations going on in the area, so the area was probably in a drought when the storm hit. The basin is so steep and the storm was so intense that infiltration was minimal.

Manning's "*n*" values selected for the upstream reach are in the range 0.055 to 0.060 and probably are low for a reach this steep. Roughness coefficients for the downstream reach were in the range of 0.030 to 0.040 and probably are reasonable. Froude numbers ranged from 1.13 to 2.42, so flow was supercritical and probably very unstable.

Possible sources of error: The downstream slope-area measurement was made in a reach that had been filled in to create a level planting area for an orchard. The fill was eroded, and it is impossible to know the channel geometry at the time of the peak discharge. The cross-section geometry could have developed after the peak discharge when flow duration was long enough to cause extensive erosion. Roughness coefficients and hydraulic computations for extremely steep basins like this one are always questionable.

The drainage area is so small that any error would have a significant effect on the unit discharge. The downstream reach may have included runoff from a left-bank tributary, but it is hard to believe the field crew would not have noticed this if it occurred.

The field review team looked for evidence of a debris flow but nothing definite was found. The flow carried a lot of sediment and could have been hyperconcentrated.

Recommendations of what could have been done differently: There is not much that could have been done differently in this basin. Local residents could have been interviewed to try to determine when erosion occurred. The upstream basin could have been investigated for debris-flow evidence or remnants of temporary dams from landslides. The contributing area may have been more accurately delineated on aerial photographs.

Site visit and review: The site was visited on April 24, 2003, by John Costa (USGS Office of Surface Water), Bob Jarrett (USGS National Research Program), Mike Nolan (USGS Western Region Surface-Water Specialist), Glenn Hess and Jim O'Conner (USGS Oregon Water Science Center), John England (Bureau of Reclamation), Joe Weber (Federal Emergency Management Agency), Gary Gallino (USGS retired), and Bill Taylor (USGS Washington Water Science Center).

The review team looked for evidence of debris flows and landslide dams in the basin, and none were identified. The team also tried to determine if inflow from a left bank side channel contributed to flow in the downstream reach, but evidence was inconclusive.

Recommendations: The original peak discharge of 903 ft^3/s should be accepted as published (rounded to 900 ft^3/s) and the rating should be downgraded to "poor."

There is a temptation to discount the discharge computed for the downstream reach, but there are no new data to justify ignoring this computation. There is no way of knowing if the after flood channel geometry is the same as the geometry at peak discharge. This is a common problem with indirect discharge measurements.

Figure A233. View upstream of lower reach of Wenatchee River tributary near Monitor, Washington, 1956. Section B at top of falls. Section A upstream of large rock.

Figure A234. View upstream of sections A and B of lower reach of Wenatchee River tributary near Monitor, Washington, 1956.

Figure A235. View upstream of sections A and C of upper reach of Wenatchee River tributary near Monitor, Washington, 1956. Section C at blue bucket.

Figure A236. View upstream of sections A and B of lower reach of Wenatchee River tributary near Monitor, Washington, 1956.

Figure A237. View downstream of headwaters, Wenatchee River tributary near Monitor, Washington, May 2003.

Figure A238. View upstream toward headwaters, Wenatchee River tributary near Monitor, Washington, May 2003.

Figure A239. View downstream toward damaged house at mouth of basin, Wenatchee River tributary near Monitor, Washington, May 2003.

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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01578310 Susquehanna River at Conowingo, Maryland

(Gaging station in Susquehanna River basin, USGS Maryland Water Science Center)

Review of peak discharge for the flood of June 24, 1972

Location: This flood was located about 3 mi north of Havre de Grace, Maryland, on the Interstate Highway I-95 bridge at 39.5812N and 76.1059W.

Published peak discharge: The peak discharge, as published in NWIS, is 1,130,000 ft^3/s for this site and occurred on June 24, 1972. Footnotes state that the peak is affected by regulation and diversion. The peak discharge and date agree with those listed in Costa (1987a, 1987b).

Drainage area: 27,100 mi^2.

Data for storm causing flood: The following quotation was taken from a Web site prepared by the Maryland Water Science Center (http://md.water.usgs.gov/floods/Agnes/Conowingo/index.html).

> "In June 1972, Tropical Storm Agnes produced significant precipitation over much of the Middle Atlantic States, particularly in the Susquehanna River Basin. Although the storm itself was only a minor hurricane, it's large areal extent and sustained path over parts of New York and Pennsylvania resulted in 6 to 10 inches of rainfall throughout the Susquehanna River Basin from June 19 to 23, with the Mahantango Creek watershed north of Harrisburg receiving as much as 18 inches.
>
> Because of the excessive rainfall and relatively wet antecedent conditions, the Susquehanna River experienced the greatest flooding known since as far back as 1784, with peak flows exceeding a 100-year recurrence interval from about the New York state line to its mouth at the Chesapeake Bay."

Photograph of bridge where measurement was made is shown in figure A246.

Method of peak discharge determination: The peak discharge is based on a current-meter measurement made at a stage of 36.76 ft, which is 0.06 ft less than the peak stage of 36.83 ft. The measurement was made at Interstate Highway 95, about 6.5 mi downstream of the gaging station. The gaging station is located at Conowingo Dam.

Adjustments were made to the measured discharge for change in stage and local inflow between the measuring site and the gage site. These adjustments were very small, amounting to a net change of -3,300 ft^3/s (-0.3 percent).

A detailed review was made of the current-meter measurement. All depths are sounded depths, and all mean velocities of verticals are based on the 0.2/0.8 method. All point velocities and mean velocities are rounded to tenths of a foot per second. A total of 24 subsections was used. Considering the total width of the channel of 4,290 ft, the average width of the subsections was almost 200 ft, with some subsections exceeding 200 ft. The channel is deep (60 ft) on the left side and more shallow (20 ft) on the right side. Velocities are distributed relatively uniformly, with the highest velocities on the left side. Subsection discharges are considerably higher on the left side. This would be the main criticism of the measurement. It would have been better if subsections on the left side were not as wide. However, there are no subsections with discharges exceeding 10 percent of the total discharge.

Depths in the deeper part of the channel were computed by applying a vertical-angle correction to determine the wet-line correction. Air-line corrections were not made because a tag was used on the suspension cable at a distance of 30 ft above the meter. Vertical angles were not recorded, or if they were they cannot be discerned in the measurement notes. Only the wet-line correction, to the nearest foot, is shown in the measurement notes. The procedures for determining the wet-line vertical-angle corrections for depth and meter positioning are not shown in the computations.

The rating curve for this site is controlled by Conowingo Dam. All measurement gage heights greater than 1,000 ft^3/s were adjusted by -6.00 ft (log offset), resulting in a straight-line rating throughout. This rating curve has a slope of 2.4, which is indicative of a section control (Conowingo Dam). Although there are very few measurements during the 32-year period 1968–2000, all measurements fit closely to the defined curve. The measurement for the 1972 flood is higher than any previous measurement by a factor of 3.2, so this measurement represents a very significant extension of the rating.

The slight extension (0.06 ft) of the rating from the measurement to the 1972 peak stage did not change the measured peak discharge because of rounding. The measured discharge of 1,128,000 ft^3/s, rounded to 1,130,000 ft^3/s, also is the published peak discharge.

Possible sources of error: The most likely sources of error in the current-meter measurement would be (1) the rather wide subsections in the deep part (left side) of the channel and (2) the vertical-angle corrections for depth and meter positioning. However, because velocities and depths are uniform, the error for subsection width probably is not significant. Errors resulting from vertical-angle corrections cannot be determined. It must be assumed that the streamgagers were familiar with vertical-angle corrections and applied them correctly.

The discharge measurement site (Interstate Highway I-95) is only about 3 mi upstream of the mouth of the river at Chesapeake Bay. Tidal fluctuations would no doubt have an effect on river flow at I-95, but because of the very high river flow, it is unlikely that tide affected the measurement.

Recommendations of what could have been done differently: The fact that a current-meter measurement was made so near the peak stage is highly commendable. The only thing that should have been done differently was to have made more detailed notes regarding the computation of vertical-angle corrections.

Site visit and review: No visit made to this site.

Recommendation: The original peak discharge of 1,130,000 ft^3/s should be accepted as published, and rated good.

Figure A246. View upstream of Interstate Highway I-95 bridge across Susquehanna River where current-meter measurement was made of June 24, 1972 flood, Susquehanna River at Conowingo, Maryland.

03611500 Ohio River at Metropolis, Illinois

(This station is operated by the USGS Kentucky Water Science Center)

Review of peak discharge for the flood of February 1, 1937

Location: This flood was located under the Interstate Highway I-24 bridge south of Metropolis, Ill., at 37.1344N and 88.6859W.

Published peak discharge: The peak discharge, as published in NWIS, is 1,850,000 ft^3/s and occurred on February 1, 1937. Footnotes state that the peak is a maximum daily average and that it is affected to an unknown degree by regulation and diversion. The peak discharge and date agree with those listed in Costa (1987a, 1987b).

A very detailed station analysis written for January and February 1937 contains a statement about flow regulation as follows:

> "Crest discharge at Metropolis was decreased about 32,000 second-feet by storage in Norris and Wheeler Reservoirs on Tennessee River, as reported by Tennessee Valley Authority."

Drainage area: 203,000 mi^2.

Data for storm causing flood: Abnormally intense rains began falling in the upper Mississippi and Ohio Valleys in December 1936 and continued through January 1937. Runoff resulted in one of the greatest floods in hundreds of years. The Ohio River was at flood stage for 1,000 mi between Pittsburg, PA, and Cairo, Ill., for a week. About 90 percent of Gallatan County, Ill., was reported to be underwater (Hoyt and Langbein, 1955). Aerial photograph of Ohio River at bridge where measurement was made is shown in figure A247.

Method of peak discharge determination: The peak discharge is based on almost daily current-meter measurements made before, during, and after the peak discharge. For this review, measurements 176 through 202 (26 measurements), made during January 14 through February 18, 1937, were available for review.

The flow of the Ohio River during this period was divided into two channels: (1) the main channel that flows adjacent to Metropolis, Ill., on the right bank and Paducah, KY, on the left bank and (2) an overflow channel that carries Ohio River flow during extreme peaks such as the 1937 flood.

The main channel discharge was measured by current meter from the Metropolis railroad bridge where the gaging station is located. Flow in the center part of the main channel was deep (90 ft maximum during the peak) and had moderate velocities (exceeding 10 ft/s in some verticals). Consequently, it was not possible to make depth soundings in the center part of the main channel where approximately 88 percent of the total flow occurred. The discharge measurement notes are not clear as to how depths in this part of the river were obtained during the time of the actual measurement. Most probably, they were based on soundings taken at an earlier or later time when velocities were low enough to permit depth sounding. It is clear, however, that the center part of the main channel flow was recomputed at a later time and that the recomputations are based on a standard cross section defined by four discharge measurements made between February 18 and 27, 1937. Spot checking indicates that the depths used in the original measurement range from about -6 to +10 ft from those used in the recomputed measurement. The cross-section area for the recomputed part of the measurement is 3.7 percent less than that for the same part of the original measurement.

Velocities in the main channel, where depth soundings could not be made, were measured at 0.2 depth, using the original depths as the basis for computing the meter settings. A factor of 0.92 was used to adjust the surface velocities for this section of the channel. This adjustment factor was based on a number of vertical-velocity curves defined at a river stage about 12 ft lower than the peak stage. In addition, a number of velocity observations made at 0.2 and 0.8 depths at a lower stage also were used to verify the 0.92 coefficient. All velocity coefficient data were defined by the same four measurements listed previously for the standard cross section.

Flow in the overbank sections on either side of the deep part of the main channel was measured directly with actual depth and velocity soundings. This was a small percentage of the total peak discharge.

All discharge measurements were computed using the "mean-section" method, which was standard practice prior to 1950. Use of the current "mid-section" method probably would make little difference in the final results.

The overflow channel was measured by current meter from a boat at a cross section located a short distance downstream of what is now Interstate 24. During the peak discharge, the overflow channel carried only about 4 percent of the total discharge.

The overflow channel is known as an "ancient" channel of the Ohio River. Water from the Ohio River spilled into the overflow channel at Golconda, about 35 mi upstream of Metropolis, and re-entered the Ohio and Mississippi Rivers near their confluence near Cairo. A map (fig. 34 in the main body of this report) scanned from an old report of the 1937 flood shows the overall configuration of the overflow channel.

The overflow was only 4 percent of the total flow and was measured almost daily by boat at a cross section near New Columbia just north of Metropolis (see fig. 34). Depths and velocities for the overflow were not excessive, and this part of the peak-flow measurement should not introduce significant errors.

The rating curve for the Metropolis gage is affected by backwater at medium and high stages and in 1937 was defined by frequent discharge measurements and based on a relation between two gages using a stage-ratio method. This method purportedly allowed for changes in water-surface slope. However, because current-meter measurements were made throughout the range of flow for the 1937 flood, the rating curve did not play a significant part in determination of the peak discharge.

Possible sources of error: The most likely source of error in the current-meter measurements would be errors in depth in the deepest part of the channel. The center part of the peak-flow measurement is based on depth measurements made about 3 weeks after the peak and at a stage at least 12 ft lower than the peak. Condition of the streambed during the peak is not known, and it is possible that there could have been significant scour occurring during the peak. A spot check of depths on both sides of the center part of the main channel, where actual soundings are available, indicates the possibility of some scour. These checks were made for measurements before, during, and after the peak (measurement nos. 181–182, 184–187; measurement no. 184 was the peak measurement). In almost every check, the standard cross-section depth was less than the sounded depth. The maximum difference was 6.7 ft for one vertical, and all other differences were less than 5 ft. The center section is about 2,600 ft wide, so assuming an average scour of 5 ft, the additional area would have been about 13,000 ft^2, which is about 7 percent of the total area of the center section.

If there are errors in depth, then there also are errors in setting the meter for the 0.2-depth velocity sounding. This would result in errors in velocity, but because the shape of the vertical-velocity curve is reasonably vertical in the upper range, the error in velocity should be minimal.

Recommendations of what could have been done differently: There are no recommendations for this site. Almost daily discharge measurements, using the best available equipment and techniques, are the best that can be done.

Site visit and review: No site visit was made. Detailed reviews of 26 current-meter measurements were made.

Recommendation: The original peak discharge of 1,850,000 ft^3/s should be accepted as published, and rated good.

In this case, the peak discharge is published as a mean daily rather than an instantaneous value. For a long-duration high peak, which this obviously is, there should be little difference between mean daily and instantaneous peaks.

Figure A247. Aerial photograph of Metropolis, Ohio. Ohio River flows from right to left; bridge where discharge measurements were made in January and February 1937, appears in left-center of photograph.

07265450 Mississippi River near Arkansas City, Arkansas

(USGS Arkansas Water Science Center)

Review of peak discharge for the flood of May 1927 (exact date is unknown)

Location: This flood was located about 3.3 mi south-southwest of Arkansas City, Ark., at 33.5597N and 91.2317W.

Published peak discharge: The peak discharge listed by USGS in the Peak-Flow File for the 1927 flood is 2,470,000 ft^3/s and is footnoted as an estimate, affected by regulation and diversion. It also is published in the station description (NWIS) as an approximate value that "*would have occurred for the May 1927 flood if flow had been confined between levees*."

The 1927 flood generally is considered the largest known flood in the downstream reaches of the Mississippi River. There are several "estimates" and published values for the 1927 peak discharge in the vicinity of Arkansas City, Ark., that range from 2.4 to 3 million ft^3/s. As near as can be determined, there is no direct or indirect measurement of this peak. In fact, even the day of the flood is not certain. Following are some of the estimates for the 1927 flood.

2,400,000 ft^3/s	Published in the book "Floods," by Hoyt and Langbein (1955).
2,470,000 ft^3/s	Described by Major Elliott as official U.S. Army Corps of Engineers data and considered to be the maximum confined discharge.
2,472,000 ft^3/s	Published in Handbook of Applied Hydrology, by Chow (1964). The peak discharge is in a section of the handbook authored by Tate Dalrymple.
2,544,000 ft^3/s	Described by John M. Barry as an "official U.S. Army Corps of Engineers reading," but this value could not be verified according to Martin Reuss of the Corps' Office of History.
3,000,000 ft^3/s	Published by John M. Barry (1997) in "Rising Tide" as the 1927 peak discharge at the mouth of the Arkansas River about 20 or 30 mi upstream of the Arkansas City gage. Barry bases the 3 million ft^3/s discharge on several sources. One is the "Bulletin of the American Railway Engineering Association, July 1927." That report indicates that a peak discharge of 2 million ft^3/s is most commonly used but that a peak discharge of 3,250,000 ft^3/s has been estimated. Location is not specified but presumed to be at Vicksburg, MS, a considerable distance downstream of Arkansas City, Ark. Other sources include (1) James Kemper, an engineer, (2) the chief engineer of the Mississippi Levee Board, and (3) U.S. Army Corps of Engineers District engineers quoted in both New Orleans and Memphis newspapers while the flood was at its worst. At a later time, the Corps claims that 3 million ft^3/s is a design flood and that the peak for the 1927 flood was about 20 to 25 percent less. Barry chose to stick with 3 million ft^3/s.

Drainage area: 1,126,600 mi^2, of which 22,240 mi^2 is noncontributing as published by USGS in the station description (NWIS) for the gaging station at Arkansas City, Ark. Drainage area in table 1 is listed as 1,130,700 mi^2, source unknown. Drainage area on the USGS Web site and in Peak-Flow File is 1,130,600 mi^2.

Data for storm causing flood: Little published information could be found regarding the nature of the storm causing the 1927 flood (see Hoyt and Langbein, 1955, p. 370). It is presumed that intense spring rains on accumulated snow in the upstream part of the basin resulted in the large runoff. Photographs were not available for this extraordinary flood, and none were taken during the 2003 review.

Method of peak discharge determination: As best as can be determined, there was no direct or indirect measurement of the peak discharge at or near the Arkansas City gage site for the 1927 flood. The results of current-meter measurements at a location described as "at Chicot, Ark." are published for most days between April 2-20, 1927, prior to the peak discharge. This site is now known as the Arkansas City gage site; however, the discharge measurement range is believed to be about 10 mi upstream of the gage site. The maximum measured discharge during this period is 1,712,000 ft^3/s on April 20, 1927. These measurements are published by the Mississippi River Commission (1930), the State of Arkansas Geological Survey (Frame, 1950), and the U.S. Army Corps of Engineers (1997). According to these reports, the measurements were made by the Mississippi River Commission and the U.S. Army Corps of Engineers. The measurements were not available for review and presumably have been lost or misplaced.

The following footnote regarding the 1927 measurements appears in the Corps of Engineers report (1997):

> "A crevasse occurred at the Mound Landing (433.6 T) about 4 miles below the discharge range on April 21, 1927. The current increased to such an extent that the boat in use could not stem the current and observations were not secured after April 20, 1927. The maximum gage reading at Arkansas City, Ark. (436.7 miles below Cairo) was 60.4 feet on April 21, 1927; the gage reading on April 22, 1927 was 57.1 feet."

The most recent topographic maps show a location on the left bank, about 6 mi upstream of the current gage location, called "Mound Crevasse." This probably is the site of the crevasse mentioned in the preceding footnote. If so, that would put the discharge measurement range about 4 mi upstream of the crevasse and about 10 mi upstream of the current streamflow-gaging station.

Notes were published by USGS in the annual station description (NWIS) for the Arkansas City gage that state that the peak discharge would have been approximately 2,472,000 ft^3/s if the flow had been confined between the levees. The authors could find no data or information that describe how this peak discharge was computed. Considering the controversy evident in such reports as Barry's book "Rising Tides" (Barry, 1997), and Martin Reuss' (U.S. Army Corps of Engineers, Office of History) review of this book, it seems apparent that if reliable engineering computations of the peak discharge were ever made they would have been referred to or quoted as a source of the peak discharge.

The crevasse resulted in a lowering of the river stage after April 20 and an apparent increase in river discharge. Some sources give the date April 20 as the date of the peak discharge. Other sources, such as the USGS Peak-Flow File, state that the peak discharge occurred in May 1927 without specifying the exact date. Considering that the last discharge measurement made on April 20, 1927, had a discharge considerably less than the estimated peak discharge, it would follow that the peak discharge occurred on a date after April 20. The less exact date of May 1927 seems reasonable.

Possible sources of error: The most obvious problem is that no one seems to know how the peak discharge was computed. Even the record of river stage after April 20, 1927, seems to be missing. Without these data and computations, it is impossible to evaluate sources of error.

Recommendations of what could have been done differently: Better documentation and archiving of the original data and computations should have been done.

Site visit and review: No site visit was made.

Recommendation: The peak discharge is debatable because sufficient evidence does not exist to properly review the published discharge and because there is a considerable amount of published controversy regarding the peak discharge. These published reports indicate that the peak discharge could range from 2.4 to 3 million ft^3/s. Most of the publications lean toward the lower end of this range. USGS publishes 2,472,000 ft^3/s and refers to it as an estimate or approximation. Considering that the peak discharge is an estimate (or approximation), four significant figures is not warranted. Therefore, it is recommended that the peak discharge should be rounded to 2.47 million ft^3/s, or even 2.5 million ft^3/s, and continued to be considered an estimate. It also is recommended that the date of May 1927 be continued as the date of the peak discharge. The drainage area shown in the annual station description and the Peak-Flow File do not agree. This difference should be resolved.