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.

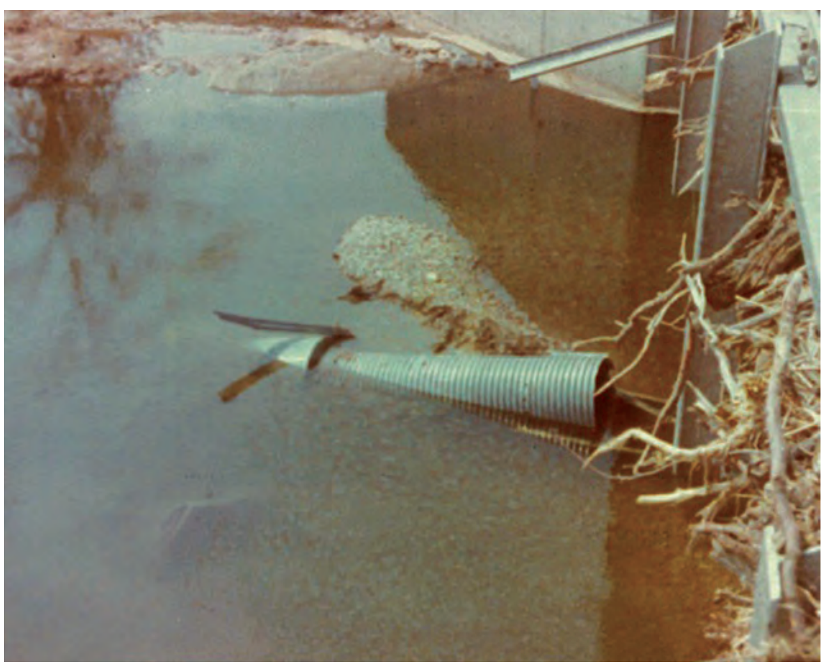

Figure A13. View of gage shelter and well taken from top of bridge, North Fork Hubbard Creek near Albany, Texas, August 10, 1978.

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.