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23	Highway Drainage
27	Bridge Design
40	Maintenance, General

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FOREWORD

This RECORD contains the proceedings of two very timely conference sessions devoted to the subject of the catastrophic floods that occurred in 1972 and the resulting impact on highway facilities. The program was so well received that the sponsoring committee was prompted to recommend publication of the proceedings. This material will be of interest to administrators, practicing engineers, and researchers interested in improving hydrologic analyses and hydraulic designs to minimize flood damage to transportation and other facilities.

Not only did the record floods caused by Hurricane Agnes cause severe damage to highways in 1972, but numerous floods of significance occurred nationwide during the year. The papers presented discuss the hydrometeorological conditions associated with these floods and the part played by state and federal agencies in such emergencies. The comprehensive report on bridge damage due to Hurricane Agnes, while not a part of the program, is included as a vital supplement to the proceedings. Debris was found to be a primary cause or a significant contributing factor to bridge damage, and scour at the substructure was found to be the other major cause. Other papers discuss recording, evaluating, and predicting extreme floods. The sessions were concluded by a panel discussion on design criteria and the need for further research in such areas as improved methods of flood prediction and risk evaluation.

—Lester A. Herr

FLOODS OF 1972

D. M. Thomas, U. S. Geological Survey

Whereas during an average year floods claim about 100 lives and $1 billion in damage, during 1972 the losses were four to five times that great. Although Hurricane Agnes caused severe flooding from North Carolina to New York, a variety of phenomena, including rainfall, snowmelt, ice jams, structural failures, and lake stages, caused flooding in Idaho, Washington, Oregon, Arizona, and other places. The floods described in this paper, all of which exceeded the magnitude of the 50-year recurrence interval, were selected to demonstrate the variety of causes.

•THE PURPOSE of this paper is to describe in general terms some of the significant floods of 1972. By any yardstick, 1972 must be known as a year of exceptional flooding. During an average year, we lose 100 lives and about $1 billion to floods; during 1972 the losses were four to five times that great.

Selecting the floods to describe was difficult, inasmuch as there was such a variety to choose from. Those selected had magnitudes exceeding the 50-year recurrence interval and some special significance. Most were selected to accentuate the variety of causes, including rainfall and snowmelt, structural failure, ice jams, and lake stages. The floods are described in chronological sequence.

The first significant flooding of the year occurred at Buffalo Creek, West Virginia, February 26, 1972. This was the largest and most destructive flood in the history of West Virginia. There were 118 lives lost, 500 homes destroyed, and 4,000 people left homeless. Property damage amounted to about $50 million, $15 million of which was damage to highway structures. The primary cause of this flood was a coal-waste dam failure. Natural runoff from rainfall and snowmelt on surrounding streams may have reached the 10-year recurrence interval. In general, the natural runoff was less than 100 cubic feet per second per square mile (cfsm) on a 1-square-mile basin and less than 50 cfsm on a 10-square-mile basin. At a point below the coal-waste dam failure, the peak discharge was about 8,500 cfsm from a 6-square-mile basin, and the discharge was at least 40 times the magnitude of a 50-year flood. The hydrologic and geologic evidence documenting this structural failure is given in U. S. Geological Survey Circular 667.

June was the month when widespread flooding really began in 1972, as shown in Figure 1. First flooding in the month started in central Washington during the period May 30 to June 3. Flooding was caused by the delayed snowmelt, after an exceptionally cool May. Discharge was the greatest since 1894 on many streams. The town of Okanogan, Washington, was inundated by 6 ft of water; otherwise, the area inundated was sparsely settled, and damage was minimal. Discharge rates on the Similkameen and Okanogan Rivers, having drainage basins from 3,500 to 7,000 square miles, were about 5 to 15 cfsm and had recurrence intervals greater than 50 years.

Between June 3 and 10 snowmelt flooding occurred in Idaho and Montana. Flood magnitudes were generally comparable to the record 1948 floods in the area, which are rated at about the 50-year recurrence interval. Unit runoff rates were 5 to 15 cfsm on very large basins such as the Clark Fork. Peak stages lasted for several days, and, fortunately, rains forecast for this period did not occur.

During the period June 7 to 10 there were three thunderstorm floods of considerable hydrologic similarity but greatly different social consequences.

On June 7, near Bakersfield, California, a thunderstorm flood caused one death and $0.25 million worth of damage. Flow rates near the center of this 500-square-mile flood area were about 500 cfsm on small basins. On June 8 in a remote area of northwest Nevada, thunderstorms hit an area about 50 miles long and 10 miles wide. Flow rates of more than 500 cfsm were observed on a 3-square-mile basin. Mud and sediment loads transported by these flows were extreme, but because the storm hit a very remote area there was no dollar damage or loss of life.

On June 9 to 10 thunderstorm floods again covering a 50- by 10-mile area struck in the vicinity of the Black Hills and Rapid City in South Dakota. Damages from this flooding exceed $160 million, with 237 lives lost and 28 persons missing. The floods damaged 2,805 houses and destroyed 770; inundated 1,305 trailers and destroyed 565; and inundated 402 businesses and destroyed 35. Discharge and runoff rates at a few selected sites are given in Table 1.

Of special interest is the effect of the failure of the Canyon Lake Dam on the peak discharge at Rapid City. Peak flows were 31,200 cfs above Canyon Lake Dam and 50,000 cfs at Rapid City below the failed dam. However, the intervening 40 square miles of drainage area produced extremely high runoff, e. g., a flood peak of 12,600 cfs from 7 square miles of Cleghorn Canyon. Because of this, it was concluded that the dam failure did not contribute a great deal to the peak flow rate at Rapid City, one of the most damaging floods in U.S. history.

The next significant flooding occurred June 18 and 19 in Westchester County, New York, and southwestern Connecticut. Peak flow rates of 70 to 170 cfsm occurred on streams draining 10 to 25 square miles. These flow rates had about a 50-year recurrence interval and are the largest on record for most sites. While this Westchester County flooding was going on, we had eyes on Hurricane Agnes along the Florida coast, but we were looking in the wrong direction.

On June 21 in the Sacramento-San Joaquin River delta, about 150 ft of dike failed. This failure caused $41 million worth of damage and necessitated the evacuation of 2,000 people, but no lives were lost. This flooding was strictly a structural failure. Upland runoff at the time was low, and tides and winds were only moderately high.

On June 21, the rains of Hurricane Agnes began in earnest, but there was one other flood of interest to be reported. That occurred on June 22 at Phoenix, Arizona, when heavy rainfall on a small basin northeast of Phoenix caused significant flooding. One stream gauger had to be rescued by helicopter. Flow from this 4-square-mile basin ruptured an irrigation ditch and caused major flooding of a large portion of Phoenix; however, no lives were lost.

During June 21 to 24 flooding was in progress from Hurricane Agnes, not in Florida, but from North Carolina to New York. In terms of people affected, extent of area damaged, and dollars lost, this is the greatest natural disaster in U.S. history. There was $3.2 billion worth of damage; one-half million people suffered losses, 116,000 dwellings and mobile homes were destroyed or damaged, 5,800 businesses were destroyed, 5,000 square miles were inundated, and 118 lives were lost. It is difficult to generalize on a storm of this size. Over one-half of the gauge sites in Maryland and Pennsylvania recorded flood peaks having recurrence intervals greater than 50 years. Flooding was extreme on both large and small streams. The Susquehanna River produced a flood peak of 1.13 million cfs (42 cfsm) from a drainage area of 27,000 square miles. And a tributary to Gunpowder River in Maryland had a peak discharge of 2,000 cfsm from a ¼-square-mile drainage basin. All in all, Hurricane Agnes caused big floods, the biggest of a big month of floods.

Areas of significant flooding during the second half of 1972 are shown in Figure 2. On July 21 to 23 heavy rains in the lake region of central Minnesota caused extensive flooding, which caused more than $17 million damage to railroads, roads, homes, businesses, and agricultural land. Flooding covered a 50-mile wide swath of poorly drained land in the Rum, Snake, and Kettle River basins. A dam failure on Knife Lake compounded the flood problems and contributed 5,000 cfs to the peak rate of 18,600 cfs at Mora. Flows exceeded the 50-year recurrence interval and ranged from less than 10 to about 45 cfsm. The greatest problem was high lake stages and slow drainage rather than flow magnitudes. Large lakes had seiches of over 1 ft, and winds caused extensive wave damage for long periods of time.

Figure 1. Areas of significant flooding during June 1972.

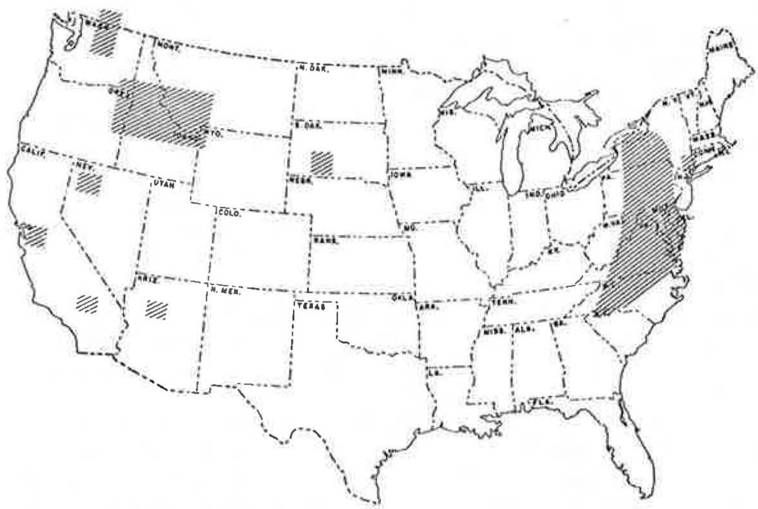
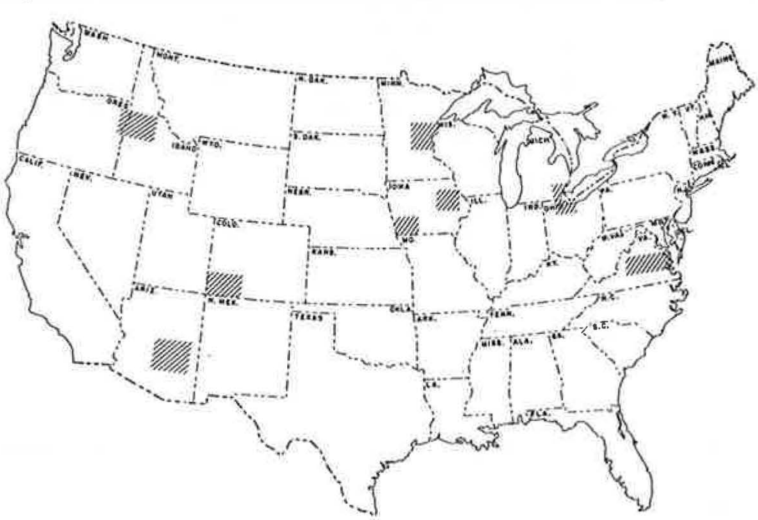


Table 1. Peak flow rates for selected sites in South Dakota for June 9 and 10.

Identification No.	Site Name	Contributing Drainage Area (square miles)	Flow Peak Rate (cfs)	Recurrence Interval (years)
–	Battle Creek at Keystone	13.6	10,800	>100
4040	Battle Creek near Keystone	66	26,200	>100
–	Battle Creek at Canyon Mouth	110	44,100	>100
4060	Battle Creek at Hermosa	176	21,400	>100
4115	Rapid Creek below Pactola Dam	1	378	
–	Rapid Creek at SD-40	8.35	5,750	>100
4125	Rapid Creek above Canyon Lake	52	31,200	>100
–	Cleghorn Canyon at Rapid City	6.95	12,600	>100
4140	Rapid Creek at Rapid City	91	50,000	>100
4215	Rapid Creek near Farmingdale	283	7,320	>100
–	Boxelder Creek at Benchmark near Nemo	37.2	1,180	7
4225	Boxelder Creek near Nemo	96	30,100	>100
–	Boxelder Creek at Nemo Road near Rapid City	117	51,600	>100

Figure 2. Areas of significant flooding during July to December 1972.



Some extensive flooding occurred in Iowa during August and September. On August 2, floods of twice the 100-year recurrence interval magnitude occurred on the Little Maquoketa River in northeast Iowa, where peak flows of 300 to 500 cfsm were recorded. On September 11 to 13 storms again hit Iowa, causing floods with magnitudes of $1\frac{1}{2}$ to 2 times the 100-year recurrence interval flood. The Little Maquoketa River again experienced flooding greater than the 100-year recurrence interval magnitude, the second such flood in a 2-month period.

As proof that more than one extreme flood can occur in a year, on October 5 to 7 there was significant flooding again in southern and central Virginia. Flows exceeded the 50-year recurrence interval for the second time in 1972 at several sites. Richmond was flooded for the third time since 1969.

Some widespread and significant flooding occurred in Colorado, Arizona, and New Mexico during October 19 to 21. Total damages exceeded $20 million, and 9 lives were lost. Flow rates of 10 to 20 cfsm from 1,000-square-mile basins set new records for peaks at sites with 40- to 50-year gauging records.

On November 14 to 15 there was some extreme flooding along Lake Erie, from Vermillion westward to Toledo, Ohio, and Monroe, Michigan, and also along the west shore of Lake Huron. The Lake Erie water surface was 2 ft above the previous known maximum and 8.4 ft above the low-water datum. Several thousand people were evacuated; 2,000 homes and businesses were flooded.

On December 4 to 15, severe ice-jam flooding struck areas of Idaho. Stages exceeded previous known maximums at places on the Snake and Salmon Rivers. The water surface on Snake River was $12\frac{1}{2}$ ft above the normal open-water surface elevation, and the towns of Weiser, Idaho, and Ontario, Oregon, were flooded. We can all sympathize with the victims of any river floods, but consider the problems of people who are flooded at temperatures of 20 deg below zero.

SUMMARY

In summary, floods during 1972 were the result of various phenomena: rainfall, snowmelt, structural failure, ice jams, high lake levels. They present many problems for hydrologists. How do we assign probabilities to a flood such as that in Rapid City? What are the probabilities of having two extreme floods such as were recorded in Iowa and Virginia? What are the consequences of extreme flooding on land use planning, and how does our knowledge of extreme floods affect structural design and inspections? These are challenging problems, but hydrologists will find that the solutions are particularly rewarding.

SUBSEQUENT INFORMATION

The Texas Highway Department called attention to severe flooding on May 11 and 12 in the Guadalupe River Basin, near New Braunfels, in south central Texas. At least 16 persons drowned, and total damage was estimated at $15 million. Unit runoff rates were extreme. A 0.48-square-mile drainage basin on Trough Creek had a peak flow rate of about 5,200 cfsm, and a 15-square-mile drainage basin on Blieders Creek had a peak rate of about 3,200 cfsm. These flow rates are among the largest ever measured in the United States.

WEATHER SITUATIONS ASSOCIATED WITH FLOODS DURING 1972

John F. Miller, National Weather Service, Silver Spring, Maryland

Floods during 1972 resulted from a variety of meteorological causes, and some were only partially the result of extreme weather events. One flood that was not caused by a major weather event occurred on Buffalo Creek in West Virginia in February; and, although precipitation of moderate to heavy amounts occurred over a fairly large area, the major flooding resulted primarily from dam failure. Of the other major flood events, one resulted from snowmelt during extremely warm periods, four resulted from more or less isolated thunderstorm convective shower activity, three were the result of widespread precipitation associated with active frontal systems, one was the result of precipitation from a tropical storm, and a final one was the result of strong winds around the Great Lakes with possibly some supplemental flooding caused by the associated rain. Several flood events are discussed.

•THE WEATHER SITUATION on the morning of February 26 (the date the dam failure occurred on the middle fork of Buffalo Creek) caused general rain from eastern Kentucky and Tennessee, northeastward across West Virginia, into New England. On the 2 previous days, a band of frontal activity had stretched generally east-west through North and South Carolina and across northern Georgia, Alabama, and Mississippi just south of the Tennessee border. A cyclonic circulation developed on this front on the afternoon of the 25th and moved east-northeast across Tennessee and the southern tip of West Virginia. On the morning of the 26th, this low-pressure system was located over the boundary between Virginia and Maryland (Fig. 1). Heavy rainfall beginning on the 24th and continuing into the 26th, plus some snowmelt, caused considerable flooding of streams in West Virginia and Kentucky. Total rainfall for this 3-day period averaged about 3 to 4 in. at several locations. This much precipitation over very small areas would have an average recurrence interval of 2 to 5 years (6). Although precipitation from this system caused flooding throughout the West Virginia-Kentucky-Tennessee region, the magnitude of the disaster at Logan County was primarily a result of dam failure. Flooding in other portions of this region was characterized as generally minor to moderate.

The temperature conditions in the north-central part of Washington during the latter part of May and early June were more common for a period in mid to late summer. A high-pressure system off and along the Pacific coast brought a flow of air off the warm Pacific Ocean, across the Cascades, and down into the interior of the state. Through the Pacific Northwest, there were generally light winds and clear skies. These conditions were conducive to bright sunshine, which pushed the maximum temperatures into the 80s and low 90s over most of interior Washington and Oregon. Figure 2 shows the daily maximum and minimum temperatures at Omak, Washington. Such temperatures were not representative of this region for so early in the summer season and resulted in relatively rapid melting of heavy snowpack over the mountains of northern Washington, which was the primary cause of the heavy flooding through the Okanogan valley region.

During the afternoon of June 7, 1972, a severe thunderstorm occurred at Bakersfield, California, with little change in the basic weather pattern. During the first week in June, there was an unusually intense and persistent upper level low-pressure center located south and southwest of southern California. The circulation around this upper level low brought great quantities of tropical moisture into the Southwest from the Gulf of California and the warm Pacific Ocean southeast and southwest of Lower California. Scattered thunderstorms broke out in the interior mountains and deserts of southern California, Arizona, and Nevada during the early portion of this period and spread northward and westward during the latter days of the first week in June.

On June 7, this moisture, mostly in the level between 5,000 and 25,000 ft, spread over the Tehachapi Mountains and into the San Joaquin Valley. At the same time, a new upper level low was moving toward the West Coast from the north Pacific. Its approach may have contributed to the vertical lift that helped to trigger the invading tropical moisture, producing large showers. In the upper level flow pattern, a very minor trough that moved around the larger low-pressure system across California, through Nevada, and toward the east during this period can be detected. The intense afternoon heating of the sun, which caused temperatures of over 100 deg during this period around Bakersfield, also helped to provide the instability in the atmosphere that promoted the intense thunderstorm development. During the morning and afternoon of the 7th, hourly weather reports showed middle and high clouds spreading northward from the Tehachapi Mountains across the southern San Joaquin Valley. There was no precipitation in the early afternoon, but shortly after 3 p.m. rain began at Bakersfield. By 3:45 p.m., thunder was heard. The wind at this time was from the south-southeast at 36 knots, with gusts to 46 knots. The intense period of rainfall lasted about 70 min, from shortly before 4 p.m. until near 5 p.m. The area covered in this storm by the 1-in. isohyet was approximately 50 square miles (Fig. 3), whereas the most intense portion, that with more than $2\frac{1}{2}$ in. of precipitation, covered only about 3 square miles. The precipitation in this storm was about six times the 100-year recurrence value for this location (3). It was also about two-thirds the probable maximum precipitation (PMP) from thunderstorms estimated by the National Weather Service. PMP is an estimate of the physical upper limit of precipitation that could occur for a particular duration and size of area (1). It is interesting to note that the weather summary for California for June 1972 (2) states: "Drought conditions continued over the State.... Seasonal precipitation was near or slightly above normal only in the northern tier counties and in the higher elevations of the Sierra...." Thus, these cloud bursts—flash-flood types of storms—can occur even with drought conditions.

The next storm to be discussed occurred in northwestern Nevada on June 8, 1972 (Fig. 4). This region is relatively unsettled, and our data on small-scale occurrences for it are probably the poorest of any of the storms mentioned in this paper. It is interesting to speculate that perhaps the same meteorological impulse that caused thunderstorm activity in the San Joaquin Valley and flash flooding conditions at Bakersfield on the previous day caused this storm. The minor trough in the upper air circulation mentioned earlier appeared to have continued to progress through a long-wave trough position off the west coast of California to a position over northwestern Nevada. This, together with the high moisture conditions that persisted throughout the entire region and in coincidence with the heating of these regions in the late afternoon, appeared again to set off the thunderstorm activity. The precipitation measured at regular weather reporting stations of National Weather Service offices and by cooperative observers did not indicate any extremely large amounts, and the population density in this region is such that bucket surveys were not conducted. Thus, there is no knowledge of the precipitation amounts in these showers. Flooding on small streams provides indirect knowledge of the existence of large showers. The regular cooperative observers and reporting stations did report some amounts near or slightly over an inch from showers on the afternoon of the 8th.

The rainfall that occurred on June 9 in the vicinity of Rapid City, South Dakota, was among the most severe precipitation and flood events to occur in the Black Hills region. Prior to the South Dakota rain, a large high-pressure system in Canada was pushing slowly southward. Early on the 9th, the leading edge of the colder air mass stretched from a weak low over northeastern Vermont, west-southwestward across southern Lake Michigan, and then westward into South Dakota.

The main feature of the low-level flow over South Dakota was its easterly direction through most of the 9th, thus giving upslope motion due to both the large-scale slope of the Great Plains and the local and pronounced terrain of the Black Hills. To the south of this cooler air mass, the prevailing weather systems were quite weak. The weakness of the systems to the south of the leading edge of the cooler air meant weak gradients and light winds. Warm, moderately moist air was characteristically present over a large region. Detailed study indicates that there was an influx of moisture near maximum conditions in a rather narrow band on the mesoscale. These conditions appear to have been an important low-level feature contributing to the heavy rainfall. The upper air flow prior to the storm shows a prominent, fairly stationary long-wave ridge over the Great Plains, with the ridge line at 500 mb (approximately 18,000 ft) just to the east of Rapid City. There was also a very weak smaller scale trough oriented northwest-southeast through southwest Wyoming. An important characteristic of these high-level charts is the prevalence of light winds through the Dakotas and westward. These light winds aloft, indicative of a lack of a strong steering current, were apparently important in keeping the massive thunderstorms in approximately the same area for several hours. At the lower levels of the atmosphere, but still above the surface, some features that could be of significance might be noted. Reflecting the surface synoptic features, the 850-mb charts (approximately 5,000 ft) show a large Canadian high-pressure system centered well north of the North Dakota border and a weak low-pressure system centered near the Colorado-Wyoming border, which moved southward in the 12 hours just prior to the storm. These two broad-scale features provided a flow of air also from a generally southeasterly direction over South Dakota. An interesting feature is that at this level the maximum low-level moisture did not extend southward over Nebraska.

An isohyetal analysis for the Black Hills storm shows several 12-in. centers of precipitation (Fig. 5). The maximum reported storm amount of nearly 15 in. fell in 6 hours near Nemo, South Dakota, about 16 miles northwest of Rapid City. All rainfall greater than 4 in. occurred on the eastern slopes of the Black Hills. The elongated, irregularly shaped, 8-in. isohyet, in general, lies between the 4,000- and 5,000-ft contours. Numerous and various-sized centers are scattered within the 8-in. isohyet. The largest, approximately 39 square miles, is about 15 miles west-northwest of Rapid City. There does not appear to be a simple or direct relation between maximum rainfall centers and terrain features of these locations. There is a slight indication that east-facing valleys may have contributed to some forced convergence of the prevailing low-level winds. The heaviest precipitation occurred in a period of about 5 to 6 hours. This heavy precipitation averaged about four times the 100-year 6-hour amount. The precipitation at the centers is also about two-thirds of the probable maximum precipitation for these locations (4).

In June 1972, there were unusually large amounts of precipitation in many portions of Arizona. This is a time of year when the probability of precipitation is relatively low in that state. Moist air entered Arizona from the south during this month with a frequency not usually prevalent until July or August. On June 21 and 22, unusually severe thunderstorm activity affected much of central and southern Arizona, with tornadoes and rain occurring in the Phoenix region. About mid-afternoon on the 21st, thunderstorms developed in a hot, moist tropical air mass over south-central Arizona, causing heavy rains and local flooding through the northeastern section of the Phoenix metropolitan area. The following morning at about 6 a.m., another severe thunderstorm system developed southwest of the Phoenix area in this relatively stagnant weather system (Fig. 6) and moved northeastward across Phoenix in the Scottsdale-Paradise valley area. This latter storm produced unusually heavy rains over a period of a few hours. The heaviest rains were about 4 to $4\frac{1}{2}$ in., which is between two and three times the 100-year rainfall value (5) and about one-third to one-half the probable maximum precipitation (4) for this region. The exact multiple or ratio depends on the location and exact duration of the rainfall at the various points in and near the Phoenix area.

During the period from June 14 through 17, a low-pressure system moved across Canada just south of the Hudson Bay, crossing the James Bay on the 15th. A cold front

Figure 1. Weather situation on February 26, 1972 (star shows approximate location of flood event; cross-hatched area indicates extent of precipitation at 7:00 a.m. EST).

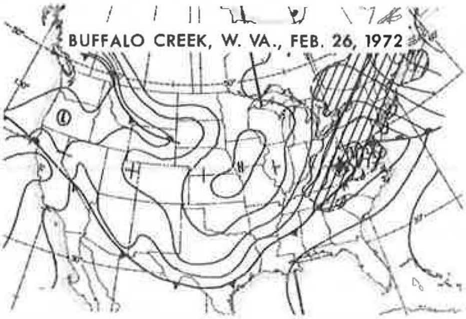


Figure 3. Total storm isohyetal map for the thunderstorm of June 7, 1972.

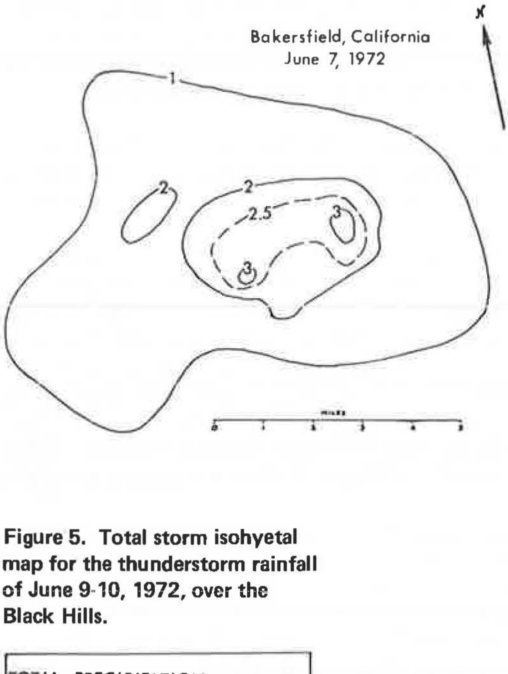


Figure 5. Total storm isohyetal map for the thunderstorm rainfall of June 9-10, 1972, over the Black Hills.

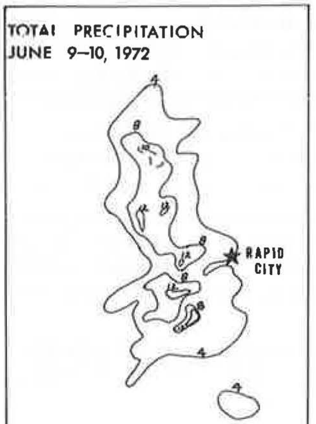


Figure 2. Maximum and minimum temperatures at Omak, Washington, during latter part of May and early June 1972.

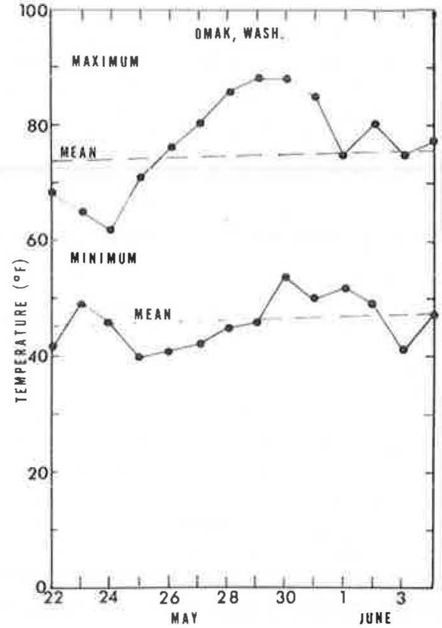


Figure 4. Weather situation on the morning of June 8, 1972 (star shows approximate location of flash flooding; cross-hatched area indicates extent of precipitation at 4:00 a.m. PST).

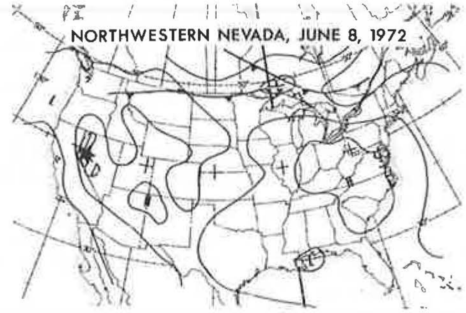


Figure 6. Weather situation on the morning of June 22, 1972 (stars show Phoenix and Hurricane Agnes at 7:00 a.m. EST).

extended southward from this system through the Great Lakes region and Indiana and into Arkansas. This frontal system continued to move eastward across the Northeast as the low moved into the Davis Straits. This frontal system passed through the New York-Connecticut area on June 17 and 18, causing considerable rainfall. Rainfall amounts reported were from 4 to 7 in. from New York City northward through Westchester County and the rest of Connecticut. Precipitation was in the form of showers on 4 consecutive days, the 16th through the 19th. The individual showers at the heaviest portion had recurrence intervals of approximately 5 to 10 years, and the total storm had a recurrence interval of about 5 to 10 years (5). One of the important aspects of this storm, as far as the northeastern portion of the country is concerned, is the wetting of the soil prior to the heavy rainfall that came from Hurricane Agnes in the succeeding days.

Hurricane Agnes formed on the 15th of June as a tropical depression off the Yucatan Peninsula. During the next 24 hours, the storm intensified and became a tropical storm. Agnes was revealed by satellite to have an unusually large circulation. On Saturday, June 17, Agnes began moving northward at about 10 mph. The following morning, hurricane-force winds were found near its center, which was some 250 miles west of the Florida Keys. By Sunday, winds were gusting from 40 to 50 mph along the Florida coast, strengthening first in the Keys and then by evening as far north as Orlando. Agnes had an unusually large circulation that brought in an easterly to southeasterly flow over Florida. As a result, winds along the east coast were often as strong as or stronger than those along the west coast. Precipitation spread over the entire Florida region.

By the afternoon of the 18th, two things were obvious: Agnes would cross the coast along the Florida panhandle, and the most destructive blow in this region would be storm tides along the west coast. Agnes moved ashore near Panama City late Monday afternoon, the 19th. It crossed northwestern Florida and weakened as it moved through Georgia. Near the 20th, the large weak depression moved northeastward across Georgia and into South Carolina. The principal effect of this storm then was rain. It was heaviest in the south in Georgia. The Carolina mountain areas were drenched, whereas in the central and coastal areas rain was light. The system continued to move northeastward across the Carolinas on Wednesday. The storm intensified again as it moved closer to the Atlantic Ocean. Cape Hatteras reported a 37-mph wind and gusts of 62 mph. Agnes reached Norfolk as a rejuvenated tropical storm on Wednesday night. It was, however, an unusual system. At one time on the 22nd, surface pressures were below 1,000 mb over an area from upstate New York to the North Carolina capes, whereas the lowest pressure hovered near 990 mb. Normal sea-level pressure is about 1,013 mb. This large region of low pressure was due in part to a quasi-stationary trough in the Ohio valley. The moisture-laden gulf air in Agnes was replenished by the Atlantic. This moist air encountered the Appalachians and triggered torrential rains over river basins from South Carolina to New York. As mentioned previously, many of these river basins were already soaked by heavy rains from the storm of June 17 and 18. Agnes moved off the Virginia capes and back out to sea late Wednesday. During the 22nd (Fig. 6), the broad system moved up the East Coast, across western Long Island, and inland near New York City. The storm became extratropical, moved westward across New York, and became nearly stationary before it turned toward the northeast again. On the 25th, it moved east-northeastward across Lake Ontario, southern New York, southern Quebec, Maine, New Brunswick, and Nova Scotia. Heavy rains continued from the 20th through the 25th over much of the northeastern United States.

The total precipitation (in inches) from Hurricane Agnes over the northeastern United States is shown in Figure 7. This is a preliminary map. It does include, through the New York-Connecticut-Pennsylvania region, some of the precipitation from the previous storm. Detailed studies of this storm are under way, but the final maps are not yet available for this period. The largest observed amounts of precipitation from Virginia through Maryland, Pennsylvania, and New York were all about twice the values for the 100-year return period where they occurred (5). The heaviest centers occurred through southeastern Pennsylvania, with regions of precipitation in excess of 16 in. Heavy rainfall centers, with precipitation depths of over 12 in., occurred from central Virginia through New York.

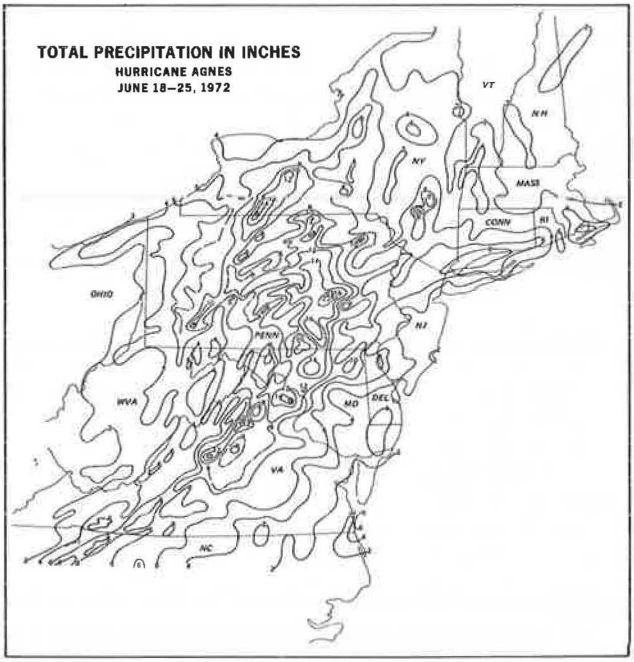


Figure 7. Preliminary total storm isohyetal map for Hurricane Agnes.

Figure 8. Weather situation on the morning of August 1, 1972 (star shows approximate location of flooding; cross-hatched area indicates extent of precipitation at 6:00 a.m. CST).

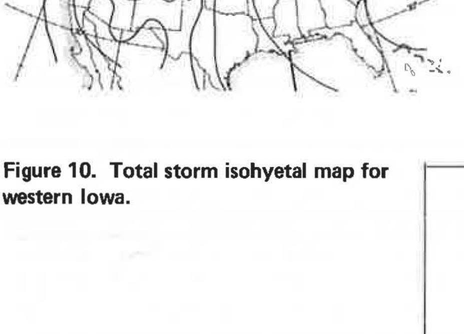


Figure 9. Weather situation on the morning of September 11, 1972 (star shows approximate location of flooding; cross-hatched area indicates extent of precipitation at 6:00 a.m. CST).

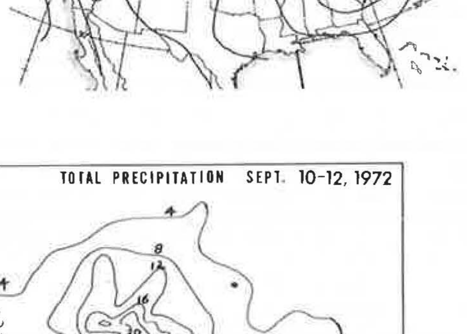
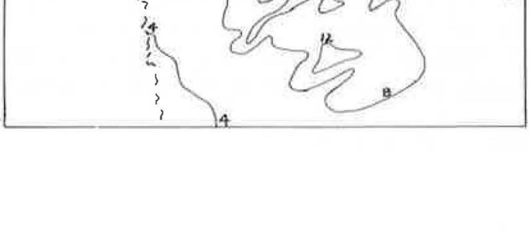


Figure 10. Total storm isohyetal map for western Iowa.



The flooding in northeastern Iowa in early August 1972 was caused by a weather system similar to that causing the flooding on June 17 and 18 in the New York-Connecticut region (Fig. 8). A low-pressure system moved across Canada through the Hudson Bay-James Bay vicinity, while a cold front trailed southeastward through the Great Lakes region into the United States. A weak low-pressure system was located just east of the Rocky Mountains, but this was not the primary cause of the precipitation. Moist air around a high centered off the southeastern U.S. coast fed moisture up across the Great Plains into the Iowa region. As the cold front moved southeastward through central United States, this warm, moist air was lifted, and numerous showers occurred. The heavy rainfall over Iowa resulted from this lifting as the cold front moved southeastward.

The situation in September 1972 was again typical of the storms that caused flooding over much of the central and eastern United States in summer and early fall. A well-organized low-pressure system moved eastward in the prevailing westerlies across Canada, while a frontal system trailed down through the United States (Fig. 9). A low developed over the eastern Colorado-western Kansas-Nebraska region on the 10th and 11th. It moved northeastward along the front, deepening as it moved. The convergence around this system and the instability in the warm, moist air feeding northward over the Great Plains were primary causes of large amounts of precipitation over Iowa. In this storm, precipitation was extremely intense. At Harland, Iowa, approximately $12\frac{1}{2}$ in. fell on the 11th, and the 3-day total was more than 20 in. The 1-day amount is about twice the 100-year value, and the 3-day total was about 2.5 times the 100-year value (6). As shown in Figure 10, the 8-in. area of precipitation is quite large. The occurrence of 8 in. in 1 day at a point in this region has a recurrence interval of approximately once in 100 years. No studies have been done that would permit an estimate of the recurrence interval of precipitation over an area this large with an average depth well over 8 in. Probably, the recurrence interval would be much greater than 100 years.

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HURRICANE AGNES—DAMAGE IN PENNSYLVANIA

Fred W. Bowser and Ming C. Tsai, Pennsylvania Department of Transportation

•THE DOWNPOUR of Hurricane Agnes was just unbelievable. Agnes sort of crept up on us. It developed near the Yucatan coast on June 19, 1972, but it was not until June 21 that it started dumping rain in Pennsylvania.

The soil at that time was nearly saturated because of the rainfalls preceding Agnes. Almost all of the precipitation turned into surface runoff immediately. The deluge of rain continued into Thursday, June 22. Maximum 24-hour totals exceeded 7 in. in a wide area of central Pennsylvania. Harrisburg had a maximum 24-hour rainfall of 12.55 in.

The Agnes flood levels of 1972 exceeded the previous record of 1936 by 3 to 6 ft in the Susquehanna River. At Wilkes-Barre, Pennsylvania, the river crested 18 ft above flood stage. The Schuylkill River in southeastern Pennsylvania crested at almost 17 ft above flood stage, which was 8 ft above the previous record set in 1902. In western Pennsylvania, the Allegheny, Monongahela, and Ohio Rivers crested at or near record levels.

The overall damage from the June 1972 floods in Pennsylvania is estimated at more than $4 billion. Damage to roads, bridges, and other transportation facilities is estimated to exceed $100 million. The flood damage estimate for state roads and bridges, based on PennDOT damage survey reports, is about $66.3 million. The damage to municipal roads and bridges is reported to be approximately $24.8 million.

On the state highway systems including federal-aid and non-federal-aid roads, 252 bridges were out of service: 80 had all spans destroyed, 17 had some spans missing, and 155 were severely damaged or had their approaches washed out. More than 370 miles of highway were closed. Of the 252 bridges damaged, 227 or nearly 90 percent were located in the Susquehanna River Basin and in four of PennDOT's 11 engineering districts.

These numbers are for the state highway system only (municipal structures are not counted). As regards municipal structures, approximately 220 bridges were totally washed out and, in addition, 150 bridges were damaged. This is a total of approximately 622 state and local bridges out of service following the June 1972 floods.

Regarding the sizes of the bridges destroyed or damaged by Hurricane Agnes, the following table gives the percentage of bridges in the state highway system that were damaged:

Length of Structure (ft)	Percentage Damaged
<10	7
10 to 19	27
20 to 49	26
50 to 99	21
100 to 199	11
200 to 399	5
>400	3

As soon as the condition of all bridges was known, the Corps of Engineers was requested to remove, from rivers and streams, all destroyed or damaged bridges that obstructed the flow and to erect a total of 18 temporary Bailey bridges throughout Pennsylvania. They have erected 14 Bailey bridges to date; the other four sites are still under study.

From the outset of the disaster, it was recognized that considerable federal aid would be required in the flood recovery effort. Therefore, as soon as PennDOT went on an emergency operation basis, the division engineer for the Federal Highway Administration in Harrisburg was contacted and arrangements were made for carrying out key FHWA functions relating to the flood relief program at PennDOT's emergency headquarters. The FHWA and PennDOT worked out the preliminary details involved in the mechanics of making the damage survey reports and in determining what damages qualified for federal aid.

By June 24, FHWA engineers were on their way to begin the job of making damage survey reports with PennDOT personnel. To get as many teams in the field as soon as possible, FHWA engineers were brought in from other division offices, regional offices, and the Washington office. Approximately 50 FHWA engineers were sent to help conduct the damage surveys on all state highways and local roads. A total of approximately 11,000 reports have been made to date. These reports form the data base for our request for federal relief funds. A total of 7,670 damage survey reports have been completed on state facilities alone.

It is our goal to have most of the replacement bridges under contract by the spring of 1973. Special instructions have been issued to our district offices to cut down on the design time. Also, the Department of Environmental Resources has given high priority to approval of waterway openings in all flood-related projects.

As of the end of December, 214 of the 252 damaged bridges had been repaired or temporary run-arounds constructed and 350 miles of the 373 miles of damaged roadway had been reopened. Also as of that date, 253 emergency contracts were negotiated for a total of approximately $15 million, and 194 have been completed to date.

No attempt has been made to review in-depth the cause of damage for each bridge. However, through the hydrologic and hydraulic reports submitted to our office for waterway approval, we did find some clues on the causes of the damage. Although the statistics are not available, the causes can generally be grouped into the following categories.

1. Scour at bridge abutments or piers due to poor foundation materials: This type of damage might have been prevented if adequate scour protection devices such as riprap protection or spur dikes had been provided.
2. Damage to bridge abutment or approach embankments due to poor upstream alignment (a sharp turn at the bridge site): In this case, some of the bridges might have been saved if they were not constructed at these troubled spots or, in the case of small streams, if channel relocations with smooth upstream alignments had been provided.
3. Debris pile-up: Some bridges lost because of debris might still remain intact if the individual span lengths of the multiple openings were adequate. Some small structures were lost by debris being carried by floodwaters and ramming the structures, e.g., mobile homes, trees.
4. Loss of approach embankments due to shift of channel courses: The shift of the channel courses was probably due to formation of sand bars in the channels and to unstable banks. This was typified in the areas where the streams were artificially widened at the bridge sites.
5. Encroachment of approach embankments or abutments into the main channels of the stream: Several bridges appear to have been lost because of this. The obstructions should not have been placed on the main channel courses.

Without question, many bridges were damaged or destroyed simply because of old-age deterioration of structural components. However, some bridges might have been saved if the hydraulic design were adequate.

The hydraulic science is a relatively new field and many areas of research remain to be addressed. I hope, under the leadership of the Highway Research Board, especially under the guidance of the Committee on Surface Drainage of Highways, more research in the field of hydrology and hydraulics can be performed so that we can design and construct hydraulically sound bridges with total confidence.

In conclusion, I would like to mention that we in Pennsylvania are battered but not beaten by Hurricane Agnes. PennDOT has received many letters of praise for the work of its personnel during the flood. It never fails that when disaster strikes there is something about Americans that makes them come through. The employees of PennDOT and other state and federal agencies proved this conclusively during Hurricane Agnes, and they can be proud of their accomplishments.

FLOOD DAMAGE IN SOUTH DAKOTA

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Larry J. Harrison, Federal Highway Administration, Denver

•SOUTH DAKOTA experienced its most devastating flood in history on June 9 and 10, 1972. The official report shows 238 dead from the flood and five people still missing. These people were swept away by the raging water. The Black Hills are very rough and steep, and the water came rushing down the steep slopes swallowing houses, cars, bridges, trucks, and everything in its path.

This flood covered portions of four counties, Lawrence, Meade, Pennington, and Custer. The rain began at 5:00 p.m. on June 9 and continued until 2:00 a.m. on June 10 when it slowed to a drizzle that continued all day. Official reports on point rainfall accumulation show up to 14 in. covering about a 40-mile long by 15-mile wide area in the eastern Black Hills area.

Damage survey estimates for local and federal-aid roads as well as for the Black Hills National Forest roads and bridges were in excess of $22 million. This included an estimated $13 million for damage to bridges alone. Two bridges were lost on the Interstate System, and emergency repairs were necessary on four others before traffic could be routed over them. Eighteen other bridges were completely destroyed on the state highway system. A total of 106 bridges were damaged or destroyed within the flood-ravaged area. Personal property losses totaled over $200 million. Most of this damage was along Rapid Creek, which flows through the center of Rapid City.

We are now reconstructing and repairing the bridges and roads in the area. Two weeks after the flood all state highways were open to traffic. Maintenance personnel and equipment were moved in from all over the state to restore the roads for emergency traffic as soon as possible. Seven emergency contracts were let to reconstruct damaged road sections for emergency traffic.

The flood peaks, given in Table 1, experienced at specific locations on six of the larger streams within the 40-mile long coverage of the June 9-10 event present a vivid picture of the magnitude of the storm from both peak flow and geographical standpoints. (All streams flow easterly out of the Black Hills.)

These peak discharges are from a U.S. Geological Survey study of the entire flood area and are subject to minor revision before a future report on the flood is issued.

When viewed from this representative sampling of extremely high flood peaks experienced throughout the flood area and because many residents of Rapid City on "sleepy" Rapid Creek had homes nestled along the banks of the low water channel and on the floodplain, it becomes clearer why so many lives were lost.

In the firm belief that we all can benefit from a review of some hydraulic design practices that were detrimental to maintaining the structural integrity of highway facilities during the passage of floodwaters, the major types of damage are briefly discussed, and recommended design considerations are presented that, if incorporated into current design practice, will lessen similar damage in the future.

Flood evidence indicates that bridges supported on spread footing foundations were much more subject to failure than those supported on steel or timber piling. Undoubtedly a combination of local scour at bridge piers, in many cases aggravated by large amounts of debris caught on the upstream face, and overall streambed scour under the bridge played major roles in the failure of structures. The tremendous forces exerted by floodwaters on the massive accumulations of debris trapped by the superstructure of bridges also contributed to the failures, as did other factors. In the instances where we had failure, these footings were founded on densely packed and cemented streambed gravels. This formation is very difficult to penetrate with piling

Table 1. Flood peaks in South Dakota for the 1972 floods.

Location	Flood Peak (cfs)	Discharge per Square Mile (cfs)	Previous Peak (cfs)
Deadman Gulch at Sturgis	4,700	800	Not available
Bear Butte Creek at Sturgis	19,500	370	12,000
Box Elder Creek north of Rapid City	51,600	440	1,180
Rapid Creek above Canyon Lake Dam	31,200	600	2,600
Rapid Creek below Canyon Lake Dam	50,000	550	3,300
Spring Creek at SD-16	21,800	210	772
Battle Creek near Hermosa	44,100	400	Unknown

and is capable of withstanding substantial spread footing loads. Spread footings founded on this formation below scour depth would be quite adequate. However, scour depths far exceeded predicted values. For these reasons, the state is seriously considering the exclusive use of pile foundations for all future designs except when spread footings are founded on scour-resistant material.

In line with our discussion of bridge foundations, we recommend that consideration be given to the use of a discharge in excess of the design quantity for foundation scour design. This would produce excess ponding depth upstream of the structure, or water would flow over the embankment, but any attendent damage would be rapidly and economically repaired. This is not so, however, should foundation support be removed by scour and the structure itself fail. We would be interested to know of any agency that does consider this aspect and would like to know the design practice used.

Typical local pier scour damage was evident in many bridges in downward displacement of the upstream portion of a bridge foundation. An unusual instance involved twin structures with the downstream bridge distressed in this manner. Debris plugged the upstream bridge with the resulting overflow impinging on the streambed just upstream of the downstream structure.

Damage to roadway embankments was widespread in that roads typically parallel streams within the floodplains. In many instances entire roadway prisms were removed. Based on the positive experience of rock and wire slope protection in areas of known high flows, the state is considering more common use of riprap where needed for design discharges with recognition that protection is afforded for the rare flood events as well.

We are all aware of existing highway-to-stream crossings that would have necessitated the expenditure of substantially less maintenance and/or flood repair money had the adverse hydraulic characteristics of the stream crossing been evaluated initially and had highway alignment been revised accordingly. The state is making a concerted effort to take this into account in current designs.

Observations of damaged stream crossings also stress the vital need for proper orientation of drainage structures parallel to natural channel alignment rather than perpendicular to roadway alignment as noted. Repair of scour on the outside as well as deposition on the inside of the resulting bend in such installations may very well require expenditure of far greater funding over the life of the project than would be occasioned with proper hydraulic design.

FLOODS IN MINNESOTA

John E. Sandahl, Minnesota Highway Department

•DURING the summer of 1972, Minnesota recorded rainfalls unprecedented in number, intensity, total rainfall, and area covered.

The mean annual total precipitation in Minnesota is 25 in., and the normal monthly total is 3.2 in. in July. The previous record 24-hour rainfall occurred in 1909 and totaled 10.8 in. The National Weather Service (NWS) indicated that a 5-in. rainfall in 12 hours has a return frequency of 100 years in central Minnesota.

The recorded 1972 storms that exceeded a 100-year return frequency are given in Table 1.

The 10 major storms experienced in Minnesota during summer 1972, in terms of total damages and fortunately in lives lost, were not so catastrophic as those in Rapid City or Pennsylvania. However, there were no mountains to trigger these storms, nor were they the result of hurricanes. The storms experienced in Minnesota could occur in any state that experiences summer thunderstorms.

According to the state climatologist, relating the number of large rainstorms that occurred in Minnesota in 1972 with respect to previous years is not possible, the reason being a recent change in the number of recording stations. The NWS has 200 official recording stations in Minnesota. Prior to 1971, unless heavy rainfall fell in the area of one of these stations, it was not recorded. In 1971, to supplement these stations, NWS began a new program with the Future Farmers of America called "Operation Rain Gauge." This program is under the technical guidance of the NWS and is administered by high school agricultural instructors. Already 1,500 gauging stations have been established in this program. The ultimate goal is to locate one station in every township. In years to come, Operation Rain Gauge will vastly improve the documentation of the location, magnitude, and frequency of these large storms in Minnesota.

The largest recorded storm in Minnesota history occurred on July 21, 1972, in central Minnesota. The storm was roughly 45 miles wide and 140 miles long, encompassing an area of 6,300 square miles. About 3,500 square miles or 55 percent of the storm area received a rainfall greater than 5 in. in the 8- to 10-hour storm duration; 178 square miles received more than 13 in. of rain.

A detailed map of this storm was prepared by NWS with data from 245 reporting stations including 215 reports from Operation Rain Gauge. These reports make this the most thoroughly documented heavy rainstorm in Minnesota's history. The storm caused the greatest monetary losses ever experienced in the state for a flash flood. Total damages are estimated at $20 million. Of this total, $5.9 million were damages to the road system, and $3.1 million of this involved damages to the federal-aid system.

Timely weather reports and forecasting coupled with actions of local, county, and state law enforcement agencies, civil defense, state and county highway departments, and private citizens saved many persons from hundreds of road washouts across the heavy rainfall area. One fatality resulted from a car being driven into a road washout. All 17 major trunk routes traversing the storm area were washed out for periods ranging from 3 to 16 days, leaving many towns isolated. Only Interstate 35 running along the extreme eastern edge of the storm area remained open. Over 4 ft of water covered the highways in many locations. There were 20 major washouts on the trunk highways, ranging from 200 to 400 ft long and 10 to 20 ft deep. In several instances, local authorities cut trenches through the highway to relieve ponding on the upstream side. In at least one instance, someone, undoubtedly on the downstream side of the highway, plugged the inlet of a culvert under the highway by making a flap gate out of a highway sign. This action contributed to a washout at the next culvert crossing downstream.

Table 1. Large rainstorms in Minnesota in 1972.

Date	Location	Rainfall (in.)
May 26	Renville County	6
May 26	Steele County	7
June 7	Martin County	8[a]
July 11	Fergus Falls, Melrose	4 to 7
July 19	Red Lake, International Falls	4 to 6½
July 21	Central Minnesota	4 to 13[b]
July 27	Aitkin and Crow Wing Counties	4 to 6
July 31	Fillmore County	4 to 5
Aug. 15	Two Harbors	5
Aug. 16	Duluth	3 to 4
Aug. 20	Duluth	3 to 4[c]
Sept. 20	Duluth	4 to 5.5[d]

[a]7-hour storm. [c]2-hour storm.
[b]8- to 10-hour storm. [d]10-hour storm.

The events that occurred at Clarissa, Minnesota, during this storm are examples of the type of problem involving highways and a village during a 13-in. rainfall. Clarissa is a typical small-town community with a population of 599. US-71 passes through the town in a north-south direction. Eagle creek has a 50-square-mile drainage area upstream from Clarissa and flows southerly along the north and east edges of town. Along the west side of town is a small draw draining about 1 square mile. US-71 crosses Eagle Creek on a bridge north of town and crosses the draw south of town on a 17-ft high embankment over a 4- by 6-ft box culvert and a 6-ft cattle pass. During the storm, Eagle Creek left its banks approximately ½ mile north of Clarissa, and the overbank flow entered the small draw causing the drainage structures under US-71 to head up 17 ft until flow over the road occurred. Backwater inundated a considerable portion of the town upstream, and the flow over the road threatened a home on the downstream side of the highway. The threat to the home was aggravated because a sight distance safety improvement project on the highway in 1960 had unfortunately moved the low point of the highway from directly over the culvert to a location 400 ft north, directly in front of the house. The highway eventually collapsed at this low point and destroyed the home.

Shortly after the storm had passed, the highway department maintenance crews delivered a 72-in. culvert to the location of the washout in order to reopen the highway. Irate townspeople demanded that the highway department construct a bridge to replace the 4- by 6-ft culvert at this location. At a meeting with the townspeople, the highway department agreed to make a hydraulic analysis of the situation and determine what the reasonable drainage needs were at this high-risk location. It was decided that a 100-year frequency discharge, including an allowance for overbank flow from Eagle Creek and proper headwater elevation controls, would be reasonable. This resulted in the selection of a 169-in. span concrete arch culvert that provided a fourfold increase in waterway opening. A public meeting with the townspeople was held to explain the reasoning for the design, which they accepted.

As in the Clarissa situation, culvert inadequacies caused major damage to both public and private property in other areas during this storm. Highway bridges over major streams and rivers generally passed the flows with minor damage. Minnesota bridge design standards provide a freeboard above the 50-year frequency design discharge. The freeboard was probably the primary factor in minimizing damage to bridges. Also it is apparent that the ponding effect of culverts and the numerous lakes and swamps in the area slowed runoff to the major streams. Examples of indirect flow measurements taken at three culvert sites receiving a 13-in. rainfall are given in Table 2. All three locations had major highway washouts.

The culvert problems resulted in the highway department receiving much unfair criticism in the news media from public officials who had requested the highway department to blow open the highways to relieve upstream ponding. The highway department has subsequently issued a written policy on this problem allowing such action provided downstream damage possibilities are assessed prior to such an action.

Table 2. Indirect flow measurements at three culvert sites.

Size and Type of Culvert	Maximum Flow Through Culvert (cfs)	Maximum Flow Over Roadway (cfs)
10- by 6-in. concrete box	705	219
36-in. diameter reinforced concrete pipe	80	135
4- by 6-ft concrete box	400	2,150

During August and September of 1972, three large thunderstorms struck the City of Duluth, a metropolitan area of 100,000 people. The storm on August 20 dumped 2 to 4 in. in 1½ hours. This storm covered 162 square miles and resulted in the most damaging flash flood in the history of Duluth, causing an estimated $12 million in damage. The severity of the washouts of the streets can be attributed to the Duluth topography, an 800-ft rise from the elevation of Lake Superior in a mile. The high velocities of flow from overflowing inadequate city storm sewers down the steep grades caused tremendous erosion. This situation was further aggravated when another storm struck on September 20 and dumped 4 to 5 in. in 10 hours. This storm caused 2 deaths and another $1 million in damage. Damage to state trunk highways in the Duluth Storms was surprisingly light except where the highways were routed over city streets.

In an effort to improve the reliability of flood flow characteristics from small (less than 50-square mile) watersheds, the Minnesota Highway Department has, for the past 14 years, been cooperating with the U.S. Geological Survey in a stream gauging program. Several of these gauge sites were in the area of the large storms. Thus, peak discharges could be computed. The gauging data recorded vary tremendously in annual peak discharges for the small number of years of record, which makes it difficult to determine a reasonable design discharge.

In conclusion, the purpose of this presentation is to attempt to illustrate the dilemma faced by the highway engineer. His task is to properly assess the public benefit versus the high economic cost factors involved in designing drainage structures for passage of runoff from large storms without adequate research information on the probable frequency, magnitude, duration, and location of these storms.

ACKNOWLEDGMENT

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OBSERVATIONS ON THE CAUSES OF BRIDGE DAMAGE IN PENNSYLVANIA AND NEW YORK DUE TO HURRICANE AGNES

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This paper evaluates the performance of bridges subjected to a major flood, determines the adequacy of design standards based on bridge performance, and recommends revisions to design standards where inadequacies are apparent. Hurricane Agnes caused severe flooding in Pennsylvania and New York, and several bridges and highways that were damaged by the floods in those states are discussed. The two major causes of bridge damage were scour at abutments and piers and impacting debris.

HURRICANE AGNES spawned the floods of June 1972, which have been called the greatest natural disaster in the history of the United States. Its impact on certain areas of the Northeast may require several years to eradicate. Although many of the bridges in Pennsylvania and New York were damaged extensively or destroyed, most survived and soon were reopened to traffic after undergoing necessary repairs and maintenance.

A few structures were less fortunate, particularly those struck by large amounts of current-driven debris and others that were subjected to extensive scour behind and beneath their abutments and at their piers. The bridge damage, however, seems remarkably slight in relation to the total number of bridges involved and the unprecedented flows. In those situations involving large streams in northeastern Pennsylvania and the southern tier area of New York, flows exceeded the largest previously experienced floods by a wide margin. For example, the June 1972 flood discharge of the Susquehanna River at Wilkes Barre, Pennsylvania, was approximately 1.5 times the magnitude of previous historic floods, which occurred in March 1865 and March 1936. Although the flood frequency of the Susquehanna's peak discharge during this record-breaking flood has not been definitely established, all of the experts seem to agree that the recurrence interval is substantially greater than 100 years. In New York, the recurrence intervals of the flooding also are much greater than 100 years at many sites.

Although the June 1972 flood was a maximum of record, many bridges survived the flood with little if any damage. These bridges were of particular interest because they obviously have features that enabled the structures to survive a severe test. The outstanding performance of these structures is a positive indication of features that constitute desirable design standards. The case history approach is used to discuss a few of the most revealing situations that were encountered as they relate to the causes of bridge damage.

DEBRIS

The most obvious cause of damage was waterborne debris that struck the bridges and collected on the superstructures and piers, as shown in Figure 1. Even in the absence of structural damage, debris removal alone was costly. At one location involving a major structure more than 1,400 ft long on the Susquehanna River in Pennsylvania, personnel of the Pennsylvania Department of Transportation and the Federal Highway Administration estimated that the cost of debris removal from the bridge deck, piers, and superstructure would approach $80,000.

Inspection of several steel bridge spans that had been carried away by the floodwaters indicated that the force of the impacting debris, in addition to, or in combination with, the pressure of the flowing water on the lodged debris, was largely responsible for this type of damage. In several instances, large portions of the piers were ripped away when the spans were dislodged from their supports. Stone piers or combination stone and concrete piers supporting multispan structures seemed to be particularly vulnerable. Considerable cracking of the pier caps also was noted, as the example shown in Figure 2 indicates.

Simply supported spans seemed to be most vulnerable to the dynamic forces produced by floodwaters and impacting debris. Bearing devices at the piers apparently did not resist the lateral forces that developed. In contrast, multispan stone and concrete arch bridges (Fig. 3) seemed to withstand these forces best, probably because of the continuity at the piers afforded by this type of construction.

In some cases it appeared that debris completely blocked the bridge openings and caused the already swollen river to increase further and inundate upstream areas. As the floodwaters progressively increased in depth, they eventually were able to overtop the bridge or its approaches. This resulted in many highway washouts and large amounts of damage to highway pavement that might not have occurred if the full capacity of the bridge opening and the river channels had been used. Adequate provision for highway-embankment overflow prevented the destruction of many bridges.

In several instances, overflow sections provided an effective means of reducing the high potential for debris damage at structures where heavy deposits of drift accumulated, clogged the bridge opening, and prevented the full hydraulic capacity of the waterway area from being used effectively. The relief provided by overflow reduced the pressure of the flowing water on the debris lodged in the bridge openings, which resulted in less structural damage than would have occurred otherwise. Some of the drift was conveyed over the roadway at the overflow sections, bypassing the bridges entirely, without being forced to enter the bridge openings. It was apparent that a judicious provision for overflow can appreciably reduce the structural damage caused by the impact of debris.

Reinforced concrete piers at the newer bridges developed a greater resistance to cracking than the unreinforced concrete, stone, or combination stone and concrete piers of the older bridges. The only exception, in this regard, was the old multispan stone arch bridges, which proved to be extremely damage-resistant. The superiority of reinforced concrete over unreinforced concrete as a construction material for piers was clearly evident.

One victim of debris was the James Street bridge at North Towanda, Pennsylvania. It was reported that the steel bracing of the trusses that supported the roadway deck was struck by at least one house trailer, possibly several buildings, a large amount of debris, and a variety of flood trash that collected on the vertical members and chords. Figure 4a shows conditions near the east abutment during the height of the flood. Note the large deflections of the vertical members of the upstream truss under the combined forces exerted by the flowing water and debris. Figure 4b shows the deflection of some of the members of the downstream truss under similar conditions. Both photographs were taken just before failure. A close examination of Figure 4b seems to indicate that at least two vertical members had been sheared from the lower chord of the truss under the action of the dynamic forces. Also note the deflection of the guardrail on the deck, which further indicates the forces on the bridge.

SCOUR

Scour at bridge abutments and piers was the second most obvious cause of damage. In many cases, the scour that occurred around bridge piers was amplified by the debris that collected on the piers. The debris increased the turbulence of the floodwater, which further increased the tendency to scour.

North Street Bridge at Wilkes Barre, Pennsylvania

With the exception of its end spans, the North Street bridge at Wilkes Barre, Pennsylvania, was totally destroyed. The piers tipped because of scour and, in some cases,

Figure 1. Debris deposited on bridge by floodwaters.

Figure 2. Cracked pier cap on Towanda Creek bridge in Powell, Penn.

Figure 3. Arch bridges, such as the Market Street bridge in Wilkes Barre, Penn., were most damage-resistant.

Figure 4. Large deflection of (a) the upstream truss and (b) the downstream truss of the James Street bridge in North Towanda, Penn.

Figure 5. Post-flood remnants of the James Street bridge.

Figure 6. Exposed piling and undermined abutment of Chemung River bridge.

completely fell over. This bridge was constructed many years ago, and the details of its substructure design are unknown. It was reported that some of the piers, but not all, had been supported by timber piling. The City of Wilkes Barre is underlain by numerous coal mines that reportedly have undergone subsidence in recent years. Mines are known to be located under the river at the bridge site.

James Street Bridge at North Towanda, Pennsylvania

The James Street bridge at North Towanda, Pennsylvania, which fell victim to debris also, was adversely affected by scour. After the Susquehanna's flow had subsided sufficiently, the Pennsylvania Department of Transportation wisely arranged for an underwater inspection of this structure. The following remarks are excerpted from the diver's report:

> Pier No. 7 can best be described as being supported by its own inertia. With the exception of a small area of not over 20 sq ft at the downstream left corner [downstream is toward the left in Figure 5], there is no ground support for the pier. The undercut at the upstream end is 40 in. high. The pilings, which in 1969 were measured at 10 to 12 in. in diameter, are now eroded to 3 in. in diameter. At least 2 piles under the upstream nose of the pier have been eroded completely in two. I was able to pass completely under the pier at a point directly below the upstream bridge truss. In this area the thickest pile measured was 5 in. in diameter. Toward the downstream end a pile 8 in. in diameter was found. I was unable to measure the piles on the interior rows downstream of the point I passed under the pier, but they felt as if the radius was larger, and the piles were definitely less spongy. The sand and gravel carried in the bed load of the river is effectively eroding away the wood in the piles.

Bridges on the Chemung River, New York

Many bridges in New York State survived the flood principally because their abutments were founded on piles. Three excellent examples of abutment scour were observed at bridges on the Chemung River. At the bridge near Lowman, although the abutment footing and a considerable length of piling beneath the footing were exposed (Fig. 6), the piling retained its load-carrying capability, and the structure remained in place in spite of the large amount of scour that occurred. With the exception of one of the abutments of the bridge near Lowman, which will be discussed subsequently in more detail, none of these structures was protected by spur dikes at the time of the flood. A careful examination of the concrete abutments failed to reveal any indication of cracking damage or other evidence of structural distress due to differential settlement or abutment rotation. After the abutments were underpinned and the washed-out sections of the approach embankments were reconstructed, the bridges were reopened.

Tioga River Bridge at Presho, New York

Another interesting situation involved a breaching of the approach embankment behind one of the abutments at a county bridge on the Tioga River at Presho. Although the footing and approximately 7 ft of piling were exposed (Fig. 7), the underlying pile foundation retained its supportive capacity and the abutment remained intact.

A short distance landward from the breach along the approach embankment, a section of the highway in a sag vertical curve was overtopped, several hundred feet of pavement were displaced, and the downward slope of the embankment was severely eroded. Water, cascading over the embankment, fell several feet to the toe of the fill and eroded the unprotected downstream slope. Although the pavement shown in Figure 8 was completely destroyed, the underlying embankment sustained surprisingly little damage. Field reconnaissance indicated that the floodplain upstream from the damaged highway carried a large quantity of overbank flow. A section of railroad track owned by Penn Central crosses the floodplain parallel to the river channel and intersects the highway within the sag. The trackage also sustained considerable damage (Fig. 8). Extensive repairs and maintenance were required before either rail or highway service could be restored.

Canisteo River Bridge at Erwins Junction, New York

Another noteworthy example of the effects of scour was provided at a site on the Canisteo River at Erwins Junction. Figure 9 shows the location of the dual bridges (indicated by the circle) and the alignment of US-15. The Canisteo River channel is located at the extreme south side of a wide valley, at the toe of steeply sloping hills. When sufficiently high flood stages are experienced, the bridge opening becomes fully eccentric in relation to the area occupied by the river and a large quantity of flow is carried by the floodplain, which is located entirely on the north bank. As shown in Figure 9, the Tioga and Canisteo Rivers meet to form the Chemung River, a few thousand feet downstream. When the Tioga discharges a large flow at this confluence, it is capable of creating backwater at the Canisteo River bridge.

At this location, US-17 borders the north side of the floodplain, paralleling the Canisteo River. US-17 is spanned by a grade separation structure about 1,700 ft north of the Canisteo River bridge at the Erwins Junction interchange. Extensive hydrologic and hydraulic design studies were made at the time this project was designed. These studies were used to design the Canisteo River bridge, which survived the flood with only minor damage to the north abutment of the upstream structure. During the June 1972 flood, the approach embankment at the north abutment was slightly scoured, and some material was eroded beneath the abutment footing, exposing about 2 ft of piling (Fig. 10). In view of the fully eccentric condition that developed and the large quantity of overbank flow that reentered the main channel at the bridge opening, it was surprising and perhaps fortuitous that the north abutment did not sustain far greater damage than the nominal amount described. The transverse movement of a large overbank flow along a highway embankment toward a bridge opening causes substantial embankment scour in the vicinity of the abutment.

Some relief for the Canisteo River bridge was provided by the grade separation structure at the interchange of US-15 and US-17 at Erwins Junction. The overflow at this interchange apparently helped to prevent more serious damage at the north abutment of the Canisteo River bridge. Perhaps the most significant factor in limiting the scour damage at the abutment and in the bridge opening was the presence of a row of large trees at the edge of the floodplain, parallel to the main channel. This line of trees extends upstream from the abutment, along the top of the riverbank, for a considerable distance. This natural feature apparently was extremely effective in preventing excessive scour damage at the abutment and in the vicinity of the bridge opening.

In contrast to the minimal damage sustained by the bridge opening, the channel downstream was extensively scoured. The north bank of the channel was lined with a dense growth of heavy trees, whereas the south bank was treeless. Both banks were subjected to large quantities of overflow. The treeless bank was severely scoured and required considerable maintenance. In comparison, the bank with trees sustained only slight scour damage because of the stabilizing influence of the heavy overgrowth.

Although the Canisteo River bridge was not overtopped, it was evident from the high-water marks on the upstream and downstream sides of the approach embankment at the bridge that its entire waterway must have been utilized. This structure survived the flooding with little damage and was never closed for repairs.

The situation at the interchange of US-15 and US-17, just north of the Canisteo River bridge, was somewhat less satisfactory. Its abutments were constructed on spread footings. Piles were not used because it was considered extremely unlikely that the river would ever flow through this interchange, and, in addition, an adequate bearing capacity for abutment support was available. It appeared that the high stages in the Canisteo were increased by backwater caused by the Tioga, and, as a consequence, US-17 at Erwins Junction interchange was inundated and the interchange structures were forced to convey a substantial flow. This flow caused a large amount of scour at the interchange abutments and resulted in substantial damage as shown in Figures 11 and 12. Both abutments were extensively undermined with the scour approximating 3 ft at some points beneath the footing and extending laterally 10 ft or more from the breast wall of the left abutment. The maximum depth of scour at the abutment shown in Figure 12 was 4½ to 5 ft. The state department of transportation felt that the structure

Figure 7. Approximately 7 ft of exposed piling beneath abutment of Tioga River bridge.

Figure 8. Approach damaged by overflow on Tioga River bridge.

Figure 9. Location and alignment of highways at bridge site. Canisteo River bridge is indicated by circle.

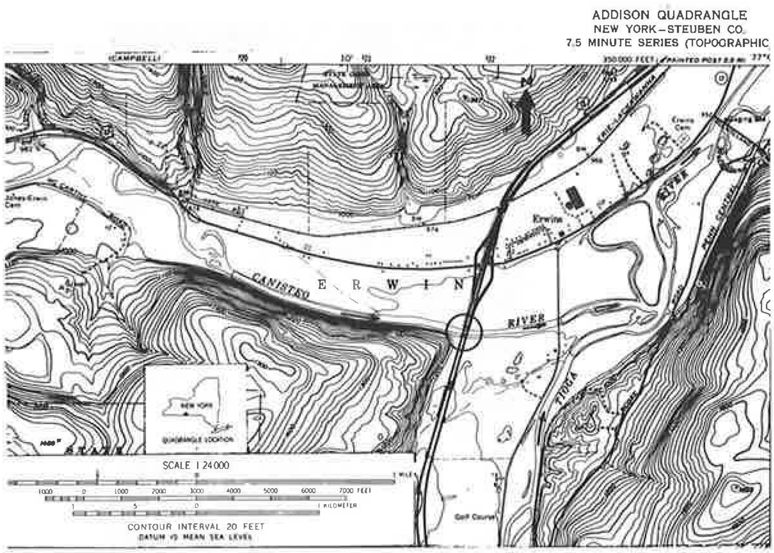

Figure 10. Upstream view of scoured bridge abutment and approximately 2 ft of exposed piling on Canisteo River bridge.

Figure 11. Timber cribbing supporting US-17 interchange bridge until abutments could be underpinned.

Figure 12. Scour beneath and behind abutments on US-15 interchange bridge.

Figure 13. Settlement due to scour on Chemung River bridge in Elmira, N.Y.

Figure 14. Rockfill spur dike near Susquehanna River bridge in Nanticoke, Penn., during 1964 flood.

Figure 15. Post-flood condition of floodplain between abutment and adjacent pier of Susquehanna bridge.

Figure 16. Remnants of earthfill spur dike near Chemung River bridge near Lowman, N.Y.

Figure 17. Enlarged scour hole upstream from Chemung River bridge in Waverly, N.Y.

survived because the steel superstructure prevented collapse. The structure was closed to traffic and braced by timber cribbing until the footings could be underpinned.

The situation provided a striking example of the importance of hydraulic and hydrologic considerations in highway design. Here were two modern bridges on the same highway separated by only a few hundred feet of roadway embankment. Both structures were designed to the same high structural and geometric standards. The major difference was that the Canisteo River bridge was designed to convey "hydraulic traffic" as well as vehicular traffic. It performed well and suffered only minor damage. In contrast the Erwins Junction interchange bridge was not designed to convey river flow; its function was to carry vehicular traffic only.

Chemung River Bridge at Elmira, New York

In those cases where the abutments were set back from the edge of the main channel, it was noted that a pier often was located near or at the top of the bank where the main channel and floodplain merge. Unfortunately a pier in this location appeared to be the most vulnerable of all to scour. Figure 13 shows a pier located near the edge of a river channel. The structure spans the Chemung River at Elmira, New York. As shown in the figure, the pier plunged into the riverbed as a result of scour and subsequent settlement. The pier apparently collected a large amount of debris, which may have increased the amount of scour. It is significant that the structure is continuous at the pier. In the author's opinion this feature was largely responsible for supporting the bridge until it could be adequately braced, thereby preventing collapse or serious damage to the superstructure due to the loss of support and settlement of the pier. The benefits of continuity in bridge structures were clearly demonstrated here.

Susquehanna River Bridge at Nanticoke, Pennsylvania

In the early 1960s, a 300-ft rockfill spur dike was constructed at an abutment of a bridge on the Susquehanna River at Nanticoke, Pennsylvania, after scour undermined an abutment and adjacent piers. Figure 14 shows the spur dike during the March 1964 flood and the turbulent conditions that existed near the dike's upstream tip where the overbank flow reentered the main channel. A comparison of the scour after two floods, both before and after the dike's construction but prior to the June 1972 flood, indicated that the dike had been effective in arresting the scour. During the flood of June 1972, the dike was overtopped by about 3 ft of water, and its effectiveness in controlling scour under such adverse conditions was severely tested. The dike remained intact and was remarkably effective during the June 1972 flood.

When the dike was examined after the flood, there was little evidence of scour other than a small, slightly eroded area near the upstream tip of the dike where a minor amount of vegetation was removed by the floodwater. There was no apparent damage along the streamward face of the dike. Figure 15 shows the area directly under the bridge near the abutment. Note the small trees growing in the area between the pier and the abutment. Because these trees were not uprooted by the floodwaters it is concluded that little if any scour occurred in this area. Prior to the spur dike's installation, the amount of scour was so extensive here that the abutment and adjacent piers were in grave danger due to undermining.

Another significant factor in the dike's excellent performance, in view of the overtopping that occurred, was the size and weight of the rock that had been used in its construction. The rock appeared to be sufficiently massive, which permitted the fill to resist the destructive effects of overtopping and to withstand the potentially damaging currents that flowed around the dike, especially near its upstream tip.

The trees and heavy brush that have grown up around the spur dike also helped in preventing the development of erosive velocities in the overbank area adjacent to the spur dike. This example illustrates that clearing and grubbing operations should be limited to that part of the right-of-way actually necessary for the construction, especially at river crossings. Full advantage should be taken of the available flow retardance of the existing vegetation. If possible, the trees and ground cover in the vicinity of the tip of a proposed spur dike should not be cleared.

Chemung River Bridge Near Lowman, New York

At a site on the Chemung River near Lowman, New York, a spur dike was constructed at an abutment to provide scour protection. A small branching channel of the Chemung rejoins the main channel near the upstream tip of the dike. The floodplain upstream from the bridge carried a large quantity of flow during the June 1972 flood. Although this earthfill spur dike was overtopped and largely washed away as shown in Figure 16, the upstream side of the approach embankment remained intact and survived the flood with no apparent damage to itself or to the vegetation growing on its slopes. Only minor amounts of erosion occurred at this abutment. In contrast, the abutment on the opposite riverbank did not have the benefit of spur-dike protection and was extensively scoured. Of the four Chemung River bridges discussed in this report, the bridge near Lowman was the only one with a spur dike at the time of the flood. (Since that time, New York has constructed spur dikes at the abutments of three of the four Chemung River bridges referred to.) The damaged spur dike was replaced at a fraction of the cost of a new structure, and reconstruction was accomplished with a minimum of traffic interruption. A new spur dike also was installed at the abutment that had been severely scoured (5).

Chemung River Bridge at Waverly, New York

At another location on the Chemung River at Waverly, New York, the floodplain on the west bank upstream from the bridge is several thousand feet wide and was inundated to a depth in excess of 6 ft. A spur dike would have provided an extra measure of safety against the possibility of scour damage, but its absence did not result in damage to the west abutment in this instance. According to the New York DOT, there was less debris at this site than at the other Chemung sites, and the west abutment was afforded some protection by its location, which is on the inside of a gentle bend in the river. Note the extensive scour hole, shown in Figure 17, at the riverbank where the floodplain and river channel join, upstream from the abutment. The embankment was intact and undamaged in the vicinity of the abutment. Because of the topography of the floodplain, a large portion of the overbank flow apparently reentered the main channel at a depression some distance upstream. A scour hole of much smaller dimensions had existed at this depression for several years. The June 1972 flood simply enlarged the existing scour hole. The most extensive damage to the bridge occurred at the abutment on the opposite bank where more than 5 ft of scour occurred beneath the footing and exposed the piling. Because the floodplain is practically nonexistent on the east side of the river, a spur dike would not have helped here.

It should not be inferred from this example that a preformed scour hole upstream from a bridge abutment is suggested as a means of protecting the abutment from scour due to overbank flow or that it is recommended in lieu of a spur dike. Floodplain topography played a major role in determining that a large portion of overbank flow would enter the main channel well upstream from the structure. In fact, the location of the scour hole may have been determined by the topography of the floodplain. But, as mentioned previously, extreme floods tend to generate complex conditions that are all but impossible to anticipate. Unfortified natural features, by themselves, cannot be relied on to always provide sufficient protection when rare and unusual conditions are experienced.

Chemung River Bridge at Chemung, New York

The quantity of scour at this site was unusual inasmuch as there was much scour and no structural damage at the west abutment and its adjacent piers. A large quantity of material was deposited in this area. The deposit extended downstream from the bridge for a considerable distance.

The east abutment was extensively scoured: Approximately 7 or 8 ft of scour occurred beneath the abutment footing. Also the highway approach embankment was breached behind the abutment. The highway bridge abutment had been connected by a levee to the abutment of a nearby upstream railroad bridge to provide scour protection. During the flood, this levee was overtopped and severely eroded, thereby enabling the floodwaters to scour the highway bridge abutment. The levee was rebuilt to a substantially higher elevation and riprapped to provide protection against a reoccurrence. At the highway bridge abutment, concrete slabs were placed in conjunction with stone riprap to provide stouter protection than could be afforded by stone riprap alone.

Lehigh Valley Railroad Bridge at Athens, Pennsylvania

Further downstream on the Chemung at the Lehigh Valley Railroad bridge in Athens, Pennsylvania, another excellent example of abutment scour occurred. It provides further corroboration of a previous statement that scour at bridge abutments largely depends on the degree of flow constriction imposed by the bridge opening. The location of the bridge and the embankment alignment are indicated by circle No. 1 in Figure 18. The east abutment of this bridge is located at the edge of the riverbank. The railway embankment traverses the floodplain diagonally, which causes a large amount of overbank flow to funnel into the bridge opening at the east abutment. Turbulence in the eddy zone, which resulted from the mixing of the overbank flow with the flow in the main channel, caused a large scour hole to form upstream. Resulting scour damage is shown in Figure 19. A spur dike would have helped to prevent scour at this abutment.

This bridge had a number of aspects that are worthy of comment. It consists of five simply supported through trusses on stone piers and abutments. The structure was obviously built many years ago. To ensure that the structure would stay in place and survive the effects of the flood in a structurally sound and usable condition required that a fully loaded freight train be driven onto the bridge to provide extra weight, or ballast, and parked on the deck until the rampaging waters of the Chemung receded. This probably saved the bridge. Nevertheless, the first pier streamward from the west abutment experienced substantial settlement during the flood and was reported to have settled 5 ft, sinking virtually straight down. Near the height of the flood, a standing wave, heavy turbulence, and a strong concentration of flow were observed in the vicinity of the pier that experienced the 5-ft settlement. This adverse flow condition may have been due in part to the large quantity of overbank flow that entered the river at the left abutment, which caused the flow in the channel to deflect toward this pier and concentrate in its vicinity.

After the flood, steel cribbing was placed around the pier for support. It was reported that an attempt was made to underpin this pier, but foundation conditions made it impossible to drive piles. The bridge is still in use although its track profile (Fig. 20) is deflected vertically in the vicinity of the sunken pier. The west abutment did not appear to have been damaged.

Another interesting aspect of the Lehigh Railroad bridge problem is concerned with the historic flood profile at the site. Figure 21 shows the water-surface profiles for several historic floods of the Chemung River in the reach where the bridge is located. The sharp break in the slope of the water-surface profile at the bridge for the 1946 flood indicates that this structure was at least a partial high-water control during the 1946 flood for a substantial reach of the river upstream from the bridge. The sharp break in slope is indicative of a large increase in flood flow velocity at the bridge. A similar condition possibly existed at the site during the recent flood and would, in that event, have been a significant factor in the amount of scour that occurred.

The location of Tozier's Bridge, indicated by circle No. 2 in Figure 18, is also shown in Figure 21. Figure 21 shows that this structure was extensively inundated by the 1946 flood; however, it was totally destroyed by the June 1972 flood. Debris, scour, and the extensive inundation combined to cause the failure in this case. The flood profiles were used in designing the bridge openings of the Chemung River bridges at Waverly and near Chemung. Their respective locations are indicated in Figure 21 by the designations D.L.& W. R.R. Bridge and proposed bridge site. The damage due to the June 1972 flood might have been much greater at both of these structures if the information provided by the 1936 and 1946 flood profiles had not been taken into account when the bridge openings were designed. It is noteworthy that the county highway bridge located immediately upstream from the structure designated on the flood profile as the D.L.& W. R.R. Bridge, the Waverly bridge, was totally destroyed by the June 1972

Figure 18. Athens, Penn.: Circle No. 1 shows Lehigh Valley Railroad bridge and approaching track alignment; circle No. 2 shows location of Tozier's Bridge.

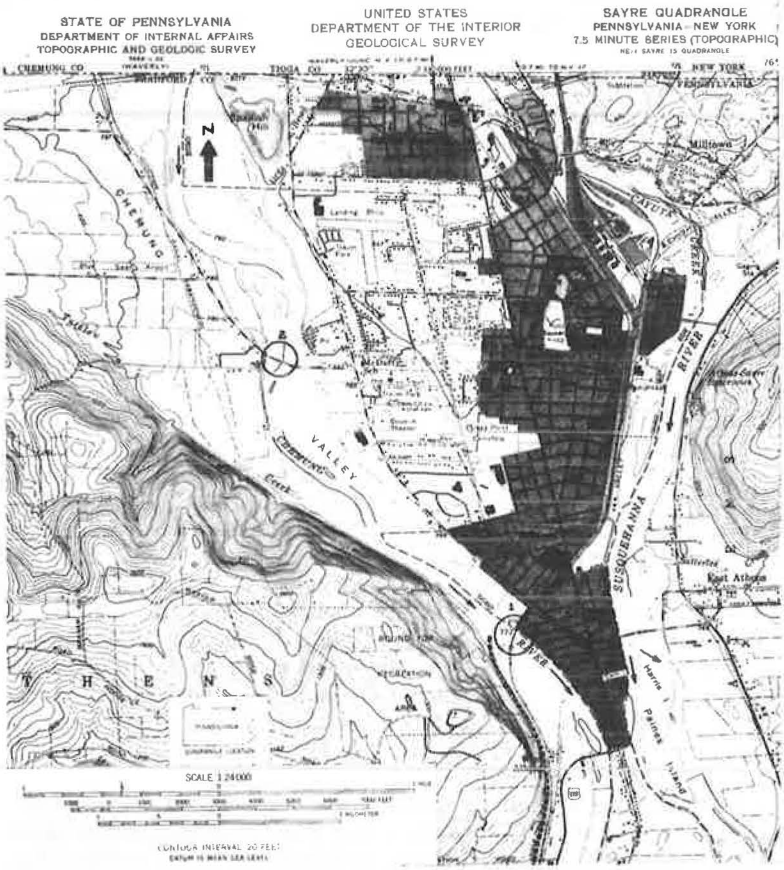

Figure 19. Upstream scour hole, eroded approach embankment, and damaged abutment of Lehigh Valley Railroad bridge.

Figure 20. Downstream view of Lehigh Valley Railroad bridge showing pier settlement.

flood. Historic flood profiles are very useful in the design of replacement and new structures.

County Highway Paralleling the Tioga River at Presho, New York

Highways that parallel streams along the top of a riverbank are vulnerable to undermining. One segment of roadway that parallels the Tioga River along the top of the riverbank at Presho, New York, was badly damaged by overflow during the June 1972 flood. The streamward side of the highway, shown in Figure 22, was extensively undermined, but the damage would almost certainly have been far greater if the embankment slope had not been heavily riprapped. The pavement consisted of 8-in. reinforced concrete with a 5-in. asphalt overlay. In one segment, the asphalt overlay was badly scoured and exposed the concrete, which in turn was extensively undermined. The damage to the pavement appeared to have been caused by trees and other debris that were carried by the stream as well as by the erosive action of the floodwaters. In situations such as these, it probably would be neither possible nor desirable to completely eliminate all damage, but a well-protected riverbank bordering the edge of the roadway in the overtopped area can help to substantially reduce inundation damages.

Susquehanna River Bridge at Dewart, Pennsylvania

Many of the Susquehanna River bridges in Pennsylvania had obviously been designed with overflow in mind. In checking bridge abutments for scour damage, many situations were encountered where overflow had occurred at some point along the approach roadway embankment. In each instance no damage was sustained by the bridge abutments. The embankment approaches seemed to have been intentionally constructed with a slight dip in their longitudinal profiles to permit overtopping when the anticipated floodwaters exceeded some predetermined elevation.

Evidently the concept of using highway embankment overflow as a means of effectively conveying flood flow was understood and successfully employed by the bridge builders of yesteryear. The wisdom of this thinking and the effectiveness of this method in preventing scour damage at bridge abutments were proved many times during this flood.

The overflow concept apparently was used in the design of the Susquehanna River crossing at Dewart, Pennsylvania. At this site the highway embankment traverses the east bank floodplain on a low profile, which gradually rises to the bridge. The downstream edge of the embankment had been paved with a concrete apron that was effective in preventing scour damage, even though the roadway had been overtopped by 7 or 8 ft of fast-flowing floodwater. An additional factor was the difference in elevation, several feet in some sections, between the crest of the pavement at the shoulder's edge and the downstream toe of the embankment. The overflow section was several hundred feet long and obviously provided considerable relief for the bridge during the flood. It appeared that the total absence of damage to the structure and its abutments was directly related to the availability of an adequate overflow section, which conveyed drift as well as floodwater. Although it was clear that drift had struck the bridge, there was no evidence of damage or structural distress. It also was apparent that the downstream slopes of highway embankments that are designed for overflow, or that are subject to overtopping, may require protection from debris and scour by paving, riprap, or some comparable type of ballasting if the expected debris is heavy and the anticipated drop between the upstream and downstream water-surface elevations is large.

The pattern of reduced damage, due to drift and scour, where overflow occurred was repeated so often at the bridge sites in Pennsylvania and New York that some consideration should be given to incorporating an overflow provision in the design criteria.

OTHER CAUSES

Bridges that were located in the proximity of bends in river channels also sustained damage. In some cases, several spans of multispan, simply supported bridges were washed away because of flow concentration in the channel near the outside of the bend. The nonuniform flow distribution contributed to the failure of the Susquehanna River bridge at Lacyville, Pennsylvania. The bend is located upstream from the structure. In traversing the bend, the flow tended to concentrate on the east side (outside) of the channel, and, as a result, the east abutment was badly scoured. The bend also contributed to a severe debris problem at this site. The two spans that were destroyed probably were lost because of the combined effects of the bend, scour, and debris.

When the performance of the bridges that had been subjected to the severe conditions was compared, it was evident that those that had been designed and built in accordance with high structural and geometric design standards generally survived the flood in better condition than those that had been designed to a lower standard. However, the June 1972 floods clearly demonstrated that designing to these high standards alone is not sufficient if essential hydrologic and hydraulic considerations are overlooked or if inadequate provisions are made to accommodate those considerations.

One bridge designed to low standards is the bridge on Schrader Branch of Towanda Creek at Powell, Pennsylvania. At this site, a concrete pier was found to be completely cracked from top to bottom over its entire cross section in the direction of its width (Fig. 23). In addition the spread footing was completely broken away from the pier shaft (Fig. 24), which gave it a wedge-shaped appearance. Investigation revealed that the pier footing apparently had been poured on the riverbed, which is composed of loose, granular material. It appeared that the weight of the bridge was primarily responsible for preventing the pier from toppling over. There was no evidence of a pile foundation beneath the footing. Lacking the support of an adequate foundation and the strength provided by steel reinforcement, the pier subsequently cracked under the applied forces and loads. Although it was impossible to determine the extent of the pier damage attributable solely to the June flood, it is likely that the pier was in a weakened condition before the flood occurred. It was evident that a previous attempt had been made to seal and grout the crack. This pier was struck by a large amount of debris, which apparently was traveling at a high velocity at impact.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

It was concluded from the examples cited that a pile foundation of sufficient length or footings founded at a sufficient depth are needed to avoid abutment or pier failure in erodible soils. Specifications in design standards giving arbitrary depths of piles for protection against scour should be avoided. Pile length or depth of spread footings should be based on the anticipated depth of scour as well as bearing capacity. In this regard, a designer should not become overly concerned that stub abutments are unduly scour-prone. A stub abutment is as acceptable as a full-height abutment if the depth of the foundation is sufficient to provide adequate structural support where extensive scour occurs. If the depth of the foundation is insufficient, the choice of abutment obviously is immaterial.

Another factor that contributed to damage was the location of abutments and piers in relation to the main river channel. During the field investigation, it was noted repeatedly that the amount of scour at bridge abutments and piers within the bridge opening depended primarily on the amount of constriction that the bridge opening imposed on the flood flow. Abutments and piers positioned near the edges of the main channel generally suffered extensive scour damage regardless of geometric shape or construction material unless they were protected by spur dikes or the approach embankments were constructed on a sufficiently low profile to provide relief as a result of overtopping.

The practice of locating an unprotected pier at the top of a riverbank should be avoided because a pier at this location is highly vulnerable to scour. If a pier must be constructed in this location, the pier shaft should extend well below the anticipated depth of scour. The outstanding performance of spur dikes during the June 1972 floods clearly demonstrated that they are one of the most effective means now available for preventing scour damage to bridge approaches, abutments, and adjacent piers in situations involving large quantities of overbank flow.

All 45 major bridges in Pennsylvania on the Susquehanna River and its major tributaries upstream from Harrisburg were checked to determine the cause of damage. Of this number only four were washed out or damaged to the extent that they had to be

Figure 21. Water-surface profiles of Chemung River for 1935, 1936, and 1946 floods.

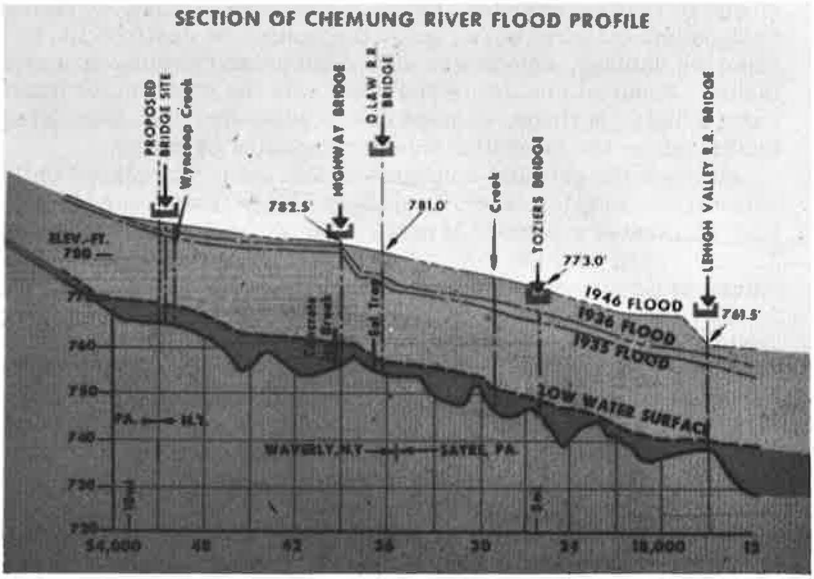

Figure 22. Undermined roadway slab and scoured embankment fill adjacent to Tioga River.

Figure 23. Cracked pier shaft supporting a county bridge on Schrader Branch of Towanda Creek.

Figure 24. Front view of cracked pier shaft and broken pier footing.

closed to traffic; however, many others were extensively damaged. (A total of 694 bridges and culverts were reported damaged or destroyed.) In virtually every case of reported damage, debris was either the primary cause or a significant contributing factor. Scour at abutments and piers was the other major cause of damage. Somewhat surprisingly, perhaps, damage due to superstructure inundation was not a significant factor unless the inundation was accompanied by debris.

Although the greatest emphasis in this study was placed on Pennsylvania's Susquehanna River bridges, several bridges in New York State were also included because they illustrated a number of noteworthy aspects of hydraulic and structural performance. In conjunction with the Pennsylvania structures they provided valuable insight into the nature of flood-induced bridge damage.

Only three bridges on the New York State highway system were reported completely washed out, but many others were severely damaged and required extensive repairs before they could be returned to full operation. [A total of 182 bridges were reported damaged or destroyed in New York State (1).] In addition, many county bridges were destroyed or badly damaged. Most of the damage was concentrated in the Genesee and Chemung River basins although other areas suffered major damage also. Debris and scour were the primary factors causing the damage—the same factors that caused the major damage in Pennsylvania.

If there was a common thread to the cause of the scour damage sustained by the bridges in Pennsylvania and New York, it could be characterized as follows: Any bridge site having an abrupt change in its cross-sectional waterway geometry, which forced floodwater to suddenly change its depth, distribution, or direction of flow, was potentially vulnerable to scour. This sudden but predictable change in a river's flood flow characteristics occurs naturally at the riverbank where the main channel joins the floodplain. Frequently, the abutments that frame the bridge opening are located there too. Often the worst scour conditions exist not at the peak of a flood but at an intermediate stage as the waters recede and the overbank flow returns to the main channel. Although scour frequently occurs in a random and unpredictable manner, a waterway constriction caused by a bridge opening in an embankment is usually most susceptible to damage in the vicinity of its abutments and piers, unless adequate scour protection is provided.

The location of the abutments and piers in relation to the main channel was an important factor in the damage that the bridges sustained. During the field investigations of the Susquehanna and Chemung River bridges it was noted repeatedly that the amount of scour at the abutments and in the bridge opening primarily depended on the amount of constriction that the bridge opening imposed on the flood flow. In general, abutments and piers that had been constructed at the tops of the riverbanks near the edges of the main channel sustained the greatest damage. This repetition of the damage pattern was prevalent, regardless of the shape of the abutments or piers and the material used in their construction. Exceptions were noted at sites where (a) spur dikes had been constructed at the abutments, (b) approach embankments had been constructed on a sufficiently low profile to provide adequate relief by means of overflow, and (c) a physical feature of the upstream floodplain had provided natural scour protection that was at least adequate for the conditions that had been experienced.

The following conclusions and recommendations were reached as a result of this study:

1. The primary causes of damage to bridges during the June 1972 flood were debris and scour.
2. In some instances, freeboard can help in reducing the damage inflicted by waterborne debris, but freeboard alone probably will not be effective in controlling debris damage when rare and unusual floods occur. Freeboard alone cannot guarantee the complete elimination of damage because the degree of protection is limited by the ever-present chance that a flood will occur that exceeds the level of protection provided by the freeboard, as well as by other engineering and economic considerations.
3. Bearing devices at piers of simply supported structures should be designed to resist dynamic flood forces, such as the horizontal forces due to impacting debris.

4. Structural continuity at bridge piers is a desirable structural feature because of the extra strength that this type of construction can provide.

5. Embankment or roadway overflow sections are recommended as a means of reducing potential debris damage at structures where heavy deposits of drift may accumulate, clog the bridge opening, and prevent use of the full capacity of the bridge waterway area. The relief provided by overflow reduces the pressure of the flowing water on the lodged debris, which in turn reduces the damage to the structure and the bridge opening. The risk of damage due to debris impacting a structure and clogging its waterway opening may be appreciably reduced because some of the drift can be conveyed through the overflow section and bypass the bridge.

6. Pile foundations for piers and abutments prevented many failures that would have occurred otherwise. According to the New York State DOT, spread footings constructed with adequate allowance for the depth of scour retained their supportive capacity also. Economic considerations might well be the deciding factor in determining which type of foundation to select in any given situation, but depth of foundation appeared to lessen the risk of failure from scour.

7. Scour at bridge abutments and within bridge openings primarily depends on the amount of constriction that the bridge opening imposes on the flood flow.

8. Piers located at the junction of the main channel and the floodplain are highly susceptible to scour. The practice of placing a pier in this location should be avoided, or more substantial provisions should be made for adequate scour protection.

9. Reliable water-surface profiles of rare floods are among the most relevant and useful items of hydrologic and hydraulic design information that can be obtained; their importance cannot be overemphasized. Bridge designers can use water-surface profiles to good advantage in a number of ways, and their use is recommended whenever possible. Although agencies such as the U.S. Geological Survey and the U.S. Army Corps of Engineers collect hydrologic and hydraulic data, state highway departments and railroads also record high-water marks from extreme floods.

10. Spur dikes were effective in controlling and preventing embankment scour behind and beneath bridge abutments and at adjacent piers even when the dikes were overtopped or partially destroyed. Their use is recommended wherever the transverse movement of a large overbank flow along a highway embankment toward a bridge opening is anticipated.

11. Highways that were designed for embankment overflow were effective in controlling scour at bridge abutments. Overflow sections might be a feasible alternative to spur dikes at some locations. Properly designed relief structures also can be used to maintain flow distribution and to reduce scour.

12. Downstream slopes of highway embankments that are designed for overflow may require protection from scour by paving, riprap, or some comparable type of ballasting if the anticipated drop between upstream and downstream water-surface elevations is large.

13. It is suggested that appropriate design committees, recommending design procedures and standards, consider incorporating provisions for accommodating highway-embankment overflow resulting from rare and unusual floods.

14. Bridges that had been designed and constructed in accordance with high structural and geometric design standards generally survived the flood in better condition than those that had been designed to a lower standard. However, the June 1972 floods clearly demonstrated that designing to such high standards alone will not be sufficient if essential hydrologic and hydraulic considerations are overlooked or if inadequate provisions are made to accommodate those considerations.

ACKNOWLEDGMENTS

Many organizations and individuals assisted me in accomplishing the field work and compiling the information on which this report is based. Hours of research were consumed in extracting pertinent information from the voluminous files of damage estimates.

Although it would be impossible to list all of the organizations and individuals who so generously contributed their information and time, I would be remiss if I failed to acknowledge the assistance provided by the departments of transportation in Pennsylvania and New York, the regional and division offices of the Federal Highway Administration for those two states, and my associates in the FHWA Washington office.

I especially appreciate the opportunity to discuss the causes of bridge damage with those federal and state highway personnel who worked to restore transportation facilities in Pennsylvania and New York immediately after the flood.

Appreciation also is expressed to the Daily Review of Towanda, Pennsylvania, for cooperation and permission to use several outstanding photographs of bridge damage that appeared in the special flood edition of that newspaper. Also, acknowledgment is made to Richard R. Church of the New York State Department of Transportation and Gerald E. Schroeder, Jack Justice, Stanley Davis, and Robert Burk of the Federal Highway Administration for furnishing several photographs.

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ROLE OF THE FEDERAL HIGHWAY ADMINISTRATION IN RESTORING HIGHWAYS DAMAGED BY DISASTERS

Thomas P. Priolo, Disaster Assistance Branch, Federal Highway Administration

Even before the disaster of Hurricane Agnes in 1972, the Federal Highway Administration had been involved in the cleanup of hundreds of disasters in the conterminous United States, Alaska, and the Virgin Islands caused by rainstorms, hurricanes, and earthquakes. Agnes almost proved to be overpowering to the Federal Highway Administration and to other state and federal agencies. The ways in which the Federal Highway Administration went about assisting the heavily damaged areas of New York, Pennsylvania, Maryland, and Virginia are described. Its involvement in the repair or reconstruction of damaged federal-aid roads and bridges is discussed.

•THE MOST DEVASTATING of all disasters occurred in 1972, and during this same year there were more disasters than in any previous year. For many years, the Federal Highway Administration has been involved in hundreds of disasters throughout the continental United States, Alaska, and the Virgin Islands. Most of these disasters were caused by severe rainstorms, hurricanes, and earthquakes and resulted in billions of dollars in damage. As long as these phenomena exist, the Federal Highway Administration will continually be involved in the repair and reconstruction of damaged roads and bridges that they cause.

In addition to involvement in disasters, under its own legislative authority, FHWA provides technical assistance and guidance to the Office of Emergency Preparedness with respect to non-federal-aid roads, streets, and bridges under the Disaster Relief Act of 1970.

Under the legislative authority of FHWA, financial assistance is provided to states under emergency relief provisions for the repair or reconstruction of highway facilities on the federal-aid system that are damaged as a result of a natural disaster over a wide area.

Congress has authorized $100 million annually for this purpose. Sixty percent of the yearly expenditures for emergency relief is appropriated from the Highway Trust Fund, and the remaining 40 percent comes from unappropriated funds in the U.S. Treasury.

Under the federal-aid highway acts, assistance cannot be provided to a state after a governor proclaims a disaster unless the Secretary of Transportation concurs in the proclamation and the state highway department makes a formal application for assistance to FHWA. When this is done, the FHWA division engineer of the state in which the disaster occurred arranges for inspection of the area; estimation of the cost of repairing or replacing the highway facility; approval of the plans, which must meet minimum AASHO standards; and inspection of the facility when work is actually under way.

When the Secretary of Transportation determines that it is in the public interest, FHWA will contribute 100 percent of all costs; otherwise, the federal contribution will be at least 50 percent.

When FHWA is requested to assist the Office of Emergency Preparedness, it provides the engineering assistance necessary to ensure that non-federal-aid roads, streets, bridges, and related structures are repaired or reconstructed in accordance with the OEP law and standards. FHWA provides the engineers to estimate the repairs or reconstruction of these damaged non-federal-aid roads or bridges, to review and approve plans, and to inspect the highway or bridge when the work is actually under way.

FHWA finds itself working with two sets of criteria (FHWA and OEP) that do not coincide. As a result of the OEP criteria, many decisions and recommendations require the judgment of the engineer in the field; however, OEP has the authority to make the final approval or recommend any changes deemed necessary.

Until Hurricane Agnes almost washed out several states, FHWA believed it dealt with some fairly large, difficult-to-manage disasters. However, Agnes proved to be the nightmare of nightmares from which participating agencies are still trying to wake up. Not a day has gone by that FHWA engineers have not had discussions with, met with, or made a trip to the field with OEP representatives regarding Agnes. Problems are still being resolved in the areas of criteria, standards, and design of roads and bridges. Congressional mail is being answered, and actions that were or should have been taken in the early stages of the disaster are being reviewed.

As an idea of the awesomeness of Agnes, comparisons will be made. The cost of the disaster, in terms of OEP federal financial participation, was equal to the 10 most devastating disasters of the past decade, and it was greater than the costs of all of the disasters of the last 5 years. To add to the magnitude of involvement, there were 47 other disasters this past year, more than double the average yearly disasters of the past. FHWA was involved in all 47.

Agnes almost proved to be completely overpowering, not only to FHWA but also to other federal and state agencies. In the past, it was routine to provide the necessary engineers and coordinate disaster operations within FHWA and with other agencies, but Agnes was so big and widespread that it was necessary to use engineers from states outside the disaster areas and an additional 30 from Washington headquarters. The total number of engineers involved in the heavily damaged states of New York, Pennsylvania, Maryland, and Virginia was 600 from the state highway departments and 300 from FHWA field and Washington offices. In Pennsylvania alone there were 11,000 damaged highway sites including more than 700 bridges that had to be inspected. In all four states there were about 25,000 damaged sites to inspect.

There were many problems encountered:

1. Many of the state and federal engineers were inexperienced with the new disaster act of 1970;
2. Initial estimates of damage were difficult to make because of inaccessibility to damaged areas because of high water, so the preliminary estimates were extremely high;
3. It was difficult to coordinate operations with state and local representatives;
4. The nonuniformity of damage evaluation and recommendations required that personal judgment be used;
5. The OEP criteria were difficult to apply to many situations encountered; and
6. The cost of estimating repairs or reconstruction varied within and between states.

As a result of these and numerous other problems, FHWA and OEP are trying to resolve problems in the areas of bridge replacement versus repair; design of bridges and applicable standards; and bridge relocation, realignment, and/or abandonment. A very critical judgment confronting the engineers was determining whether a bridge was repairable or required rebuilding. FHWA objected to the reconstruction of a superstructure on abutments that were the only remains of the bridge after the flood. Some of the minor problems eventually resolved themselves; others are still in the works.

It will cost about $250 million to repair all highways, roads, and bridges in all four states affected by Agnes.

Other involvements of FHWA included the coordination of debris removal by the state highway department from federal-aid highways and from non-federal-aid roads and streets. Agnes was so widespread and devastating that the Corps of Engineers assisted in the removal of debris from the state system and off-system roads in Pennsylvania.

FHWA and the Corps of Engineers worked together in the erection of Bailey bridges at critical river crossings where essential bridges were completely destroyed. Between 40 and 45 Bailey bridges were used at critical highway crossings. Some of these bridges will remain in place for 2 years or more.

One of the outstanding coordinating efforts by the FHWA was in the movement of 18,500 mobile homes from the disaster areas of Pennsylvania and New York. This involved working with HUD, 35 state highway departments, mobile home manufacturers, hauling contractors, and, in some instances, the governors of the 35 involved states through which mobile homes were transported; OEP coordinated the whole disaster operation.

Mobile home movement was expedited by streamlining special permit procedures; permitting nighttime, holiday, and weekend movements; lifting restrictions on load, length, and width of tractor and mobile home combinations; and lifting restrictions on the use of escorts. All this was done without compromise to safety on the highways, and no serious accidents were reported.

We are now, and will be continuously, dealing with the Environmental Protection Agency by ensuring that man and nature are disturbed to a minimum when making repairs; with HUD by ensuring that future floods and floodplains are taken into consideration in the design of new or replacement highway structures; with the OEP by striving to iron out standards, criteria, and inspection procedures so that the restoration work after the next disaster will be a smoother operation; and finally, with several congressmen in explaining why the bridge scheduled for reconstruction a year ago has not been completed and how much longer it will be until it is completed.

There was a time when we had a breather between disasters, but we now find that before one disaster is overcome another is upon us. If this past year is an indication of what the future will be, we may find that in the next 20 years the face of much of the United States will have been reshaped.

Hurricane Agnes, without a doubt, is the most devastating disaster that ever hit this country and may be the beginning of a series of great disasters that will follow.

RECORDING FLOODS AND FLOOD DAMAGE

Walter Hofmann, U.S. Geological Survey

•RATHER THAN try to cover the whole gamut of both federal and state agency responsibilities for recording floods and flood damage, I will limit this paper to the activities that are the responsibility of the U.S. Geological Survey. The fact is the Geological Survey has very little responsibility for the flood damage side of recording

As part of its basic mission, the Geological Survey has the responsibility for recording the stage, discharge, and areal extent of floods. This information provides hydrologic data for many purposes. Typical uses of hydrologic data on floods are for flood-frequency analysis, economic design of structures in and adjacent to the riverine system, land use planning and management (particularly management of the floodplain), emergency planning and evacuation, and flood warning systems. In addition, in recent years, special data needs have developed for the operation of the National Flood Insurance program. Information also is needed for post-flood rebuilding and zoning, and to an extent for damage and loss assessment.

Major floods are documented by collecting data in the field and then by reporting the data so that they are available to those who need the information. I would like to discuss these two activities separately.

The criterion for the data to be collected is that they must describe the stage and magnitude of the flood at the gauged points and at miscellaneous sites that may be of specific interest to action agencies, federal, state, and local, insofar as resources permit. Also the areal extent of the flooding should be delineated through field surveys and photogrametrically.

At gauge sites, we operate a continuous water stage record and make periodic discharge measurements, generally a relatively inexpensive procedure. However, during extreme floods, collecting records of stage and discharge can be extremely difficult. In the Hurricane Agnes floods, we lost several of the gauging stations in the area.

In addition to gauges washing out, many become inaccessible because of flooding. The bridges and roads leading to the gauge wash out, and we cannot get there in time to measure the peak discharge with a current meter. This in turn requires indirect measurements of the peak discharge, and these are expensive. They not only are expensive and time-consuming, but require experienced people to make the surveys. For the Agnes floods we detailed about 45 or 50 people from various parts of the United States who had this expertise for periods up to 3 months. For the Rapid City flood we made 19 indirect measurements and for the Agnes floods about 320. The added cost for the additional measurements and other data collection activities was about $0.5 million. This does not include the cost of replacing destroyed gauging stations.

Whereas the U.S. Army Corps of Engineers had $50 million for their Agnes flood activities, the Geological Survey was not so fortunate. Our reserves for special flood activities at the beginning of the fiscal year were $75,000. The last of the reserve was allocated for earlier floods; so, when Agnes hit, there had to be a major reprogramming and redirection of funds from the remainder of our program. This was not sufficient to do everything that we wanted to do, but I think the essential work was accomplished. We did have financial support from the Corps of Engineers for some of the things they were primarily interested in. We also obtained support from the states for the replacement of gauging stations that were in the cooperative program.

To record the areal extent of flooding, we relied principally on aerial photography. It is a fairly standard procedure that, during major floods, our district offices contract with a local aerial photography firm to fly the river as close to flood stage as possible to give us a photographic record of the extent of flooding during the peak. This works out well if the weather permits, but frequently during a flood situation there is cloud cover, rain, and heavy winds, which limit both flying and adequate photography.

We obtained aerial photography for the Rapid City flood, but this area was not flown until 2 days after the peak. The peak for this flood was quite rapid, and the photography was of little value in delineating the inundated area. Therefore, field surveys were necessary. We did get good photographic coverage for Hurricane Agnes floods in Virginia, Maryland, and southern Pennsylvania. Very little was obtained in northern Pennsylvania and New York because of weather conditions.

Another constraint on flying is the problem of air traffic. For example, we could not fly along the Potomac River in the vicinity of National Airport. Thus, the photographic coverage of the Potomac only went as far downstream as Key Bridge.

The preceding are some of the factors involved in the collection of data. Probably as important or even more important is the dissemination of the information and data collected. To do a better job, we tried to get some input from the users of our information at the time of the Hurricane Agnes floods. Geological Survey staff members went to Richmond, Harrisburg, and a few other places to talk to planners and managers to see what type of information would be most helpful to them. We did not get much response, so we are continuing to do what we have done in the past with some modifications. However, I think this group could be a real help to us by suggesting the type and timeliness of their information and data needs and also by suggesting what we could do to get our information out better and faster. There is considerable cost involved in the collection and dissemination of data, so we want to be as efficient as possible in furnishing the data needed.

In the past for major floods, we have published the information in the Water Supply Paper (WSP) series. These reports include a description of the flood, information on the meteorology, some analysis of flood-frequency relations, some comparison with previous floods, and a tabulation of stage and discharge at various time intervals covering the flood peak. The problem with the WSP publication is that it is not available until a considerable time after the event. This serves the important need for historical documentation. However, we do like to get out some quick releases at the state level; these provide the magnitude of peak stages and discharges. These are usually released 2 to 3 weeks after the event and are available at district offices.

To depict the areal extent of the flooding, we have recently attempted to produce photo mosaics based on aerial photography. These are available 3 to 4 weeks after an event and can be purchased as photocopy or as an ozalid print. For the Agnes flood our coverage of various rivers was quite good. (In the last year there has been a real effort to better coordinate the operations of the National Weather Service, NOAA, and the Geological Survey, Water Resources Division, in terms of flood activities. Some interagency committees have been formed. We have established an individual in each to serve as flood coordinator so the activities of each agency will be better coordinated.)

For the Rapid City flood we are preparing a joint report with NOAA to include the meteorological information, the magnitude and frequency of floods, and all the stream flow data in one report. Also we are publishing a Hydrologic Atlas that shows the areal extent of flooding. A joint report also is planned for the Agnes floods. This is still in the planning stage, but we are requesting our districts to assemble the information that will be included in this report.

As I said the areal extent of the Agnes floods has been reported and I doubt whether we can do much more. We have several sets of uncontrolled photo mosaics for the James, Potomac, and Susquehanna Rivers and some tributaries. The scale of these mosaics ranges from 1:5,000 to 1:12,000; these were completed in 1 month and made available to the public as they were completed. In addition to these photo maps, we prepared some flood inundation maps, directly on quadrangle sheets, based on field surveys. These will be published as hydrologic atlases.

In conclusion, I would like to reiterate that we could do a better job of providing the information you need if we had a better understanding of what those needs are, in terms of both the type of data and the timeliness of the data need. Our present program was developed in response to user comments and needs, but we always welcome additional comments and suggestions.

PROBABILITY DISTRIBUTION OF EXTREME FLOODS

Clayton H. Hardison, U. S. Geological Survey

Two aspects of the unusually large floods experienced in 1972 are discussed: how such large floods affect the computation of the 50-year peak and how these floods plot on a probability distribution of extreme floods based on analysis of 157,000 annual peaks at about 200 stream gauging stations. Weight factors are given to be used in computing 50-year peaks both with and without extreme floods included.

•FOR 10 OF THE STATIONS affected by the 1972 floods associated with Hurricane Agnes, the use of the recommended Log-Pearson Type III (LPT3) procedure with observed peaks through 1972 results in 50-year peaks that average three times as large as the 50-year peaks computed without including the Agnes floods. Even when generalized logarithmic skew coefficients were substituted for the sample skew coefficients in the LPT3 computations, the 50-year peaks at these 10 stations averaged about 80 percent larger with the Agnes floods than they did without. A modified application of a procedure proposed by Kirby (1) gives 50-year peaks at these stations that average only 33 percent larger with Agnes than without. The details of the application will not be dealt with at this time, but Figure 1 shows curves that evaluate Kirby's equations. The average skew coefficient of 2.0 used in computing the curves is reasonably close to the average skew coefficient at the gauging stations in parts 1 to 6 of the water supply papers (2).

According to Kirby, the 50-year peak computed by using the full sample should be given a weight equal to the factor shown as the ordinate in Figure 1, and the 50-year peak computed without the highest peak should be given a weight equal to one minus the factor. The factor tends to increase with the length of record and to decrease with the parameter. With 40 years of record and with the largest peak 5.5 standard deviation units above the mean, for example, the weight to be given to the 50-year peak based on the full sample is 0.2, whereas that to be given the 50-year peak based on the censored sample is 0.8.

Before such factors were used to obtain the 50-year peaks in this investigation, however, the 50-year peaks from the censored samples were multiplied by a factor greater than one to remove the bias introduced by dropping the highest peak in an ordinary sample. This factor was computed as the antilog of (1.56 - 0.8 log N) (LSD), where LSD is the average logarithmic standard deviation of the censored and uncensored samples weighted by the factor given in Figure 1. (For LSD = 0.3, the adjustment ranges from 1.15 for N = 50 to 1.43 for N = 20.) A further restriction on the weighting procedure was that, if the adjusted censored Q_{50} was greater than 95 percent of the uncensored Q_{50}, the uncensored Q_{50} was used without adjustment. In all cases, whether censored or uncensored samples, the logarithmic skew coefficients used in the LPT3 computations of Q_{50} were taken from a map of generalized skew coefficients. The generalized skew coefficients ranged from 0.5 along the east coast to -0.3 in the Midwest, to a high of 0.7 in the Great Plains, and down to -0.2 in the extreme northwestern part of the area covered by this investigation.

The probability distribution of extreme floods was investigated by considering records of annual peaks at about 200 stream gauging stations in parts 1 through 6 where the maximum peak during the period of record was at least twice as large as the highest of the other peaks. The original intention of using the ratio of the maximum peak discharge to the 50-year peak discharge as a measure of the magnitude of

Figure 1. Kirby (1) weight factors for outliers, assuming an average skew coefficient of 2.0.

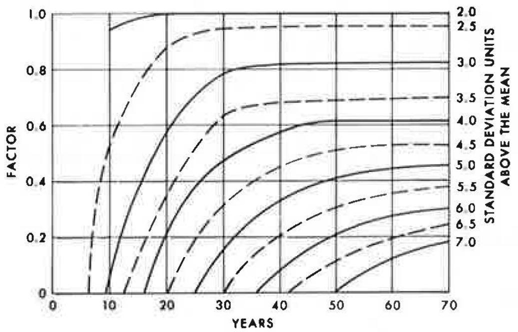
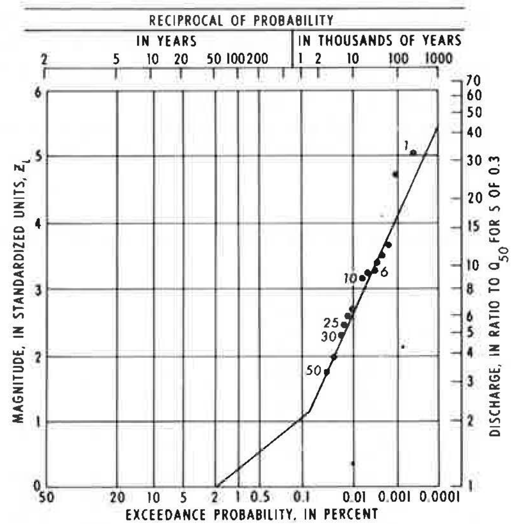


Figure 2. Frequency distribution of extreme floods given in Table 1.



extreme floods had to be modified to account for the variation in slope of the flood-frequency curves. The standardized units finally used as a measure of magnitude were computed by dividing the logarithm of the ratio by S, the slope of the flood-frequency curve at the 50-year recurrence interval. (Values of S, in log units per standard deviation unit, are given by the logarithm of Q_{100}/Q_{50} divided by 0.272.)

When ranked according to standardized units, the 50 most extreme floods in parts 1 through 6 define the probability distribution shown in Figure 2. The scale on the left is in standardized units, Z_L units, whereas that on the right is the corresponding ratio to the 50-year peak for a station that has an S value of 0.3. The numerals by the points indicate order number, inasmuch as below order number 6 many of the points have been omitted to avoid overcrowding. Exceedance probabilities are shown along the bottom. As an aid to understanding the meaning of small probabilities, the reciprocal of the probability is shown in thousands of years along the top of the figure.

The point of intersection of the line defined by these points with the line extrapolated above the 50-year flood indicates that a straight-line extension on log probability paper from the 50-year point to the 1,000-year point is legitimate but that, above the 1,400-year point, the slope of the probability distribution is 2.6 times the slope at the 50-year point.

The 50 peaks used to define the upper line are fairly well distributed throughout the area covered by parts 1 through 6, although most of them occurred in the middle- and north-Atlantic states. The floods that caused these peaks resulted from 20 different storms in 19 years. Eight of the peaks occurred in 1972, four occurred as a result of the 1955 hurricane in New England, six were the result of an August 1940 storm in North Carolina and Tennessee, and nine occurred during the 1964 flood in northwestern Montana. The three peaks with the highest Z_L values occurred during the Montana flood.

The highest ratio to 50-year peak is 14.87 for the 1965 flood at Plum Creek, Colorado, but, because of the relatively high S value of 0.397 at this station, the resulting Z_L value of 2.96 puts it in the number 13 position. In contrast, when the logarithms of the ratios of 7.89 and 9.89 for two of the Montana peaks are divided by the relatively small S values of 0.161 and 0.202, the resulting Z_L values of 5.57 and 4.93 are number 1 and 2 in this array.

Eight of the peaks result from the Hurricane Agnes floods and have ratios to the 50-year peak ranging from 2.76 to 6.26. The resulting Z_L values for these peaks have order numbers ranging from 5 to 45, which indicates probabilities of exceedance of from 1 in 3,000 to 1 in 33,000. Two peaks from the 1972 flood in the Rapid City, South

Dakota, area have ratios to the 50-year peak of about 9.5, but because of the high S values of 0.47 the resulting Z_L values of 1.80 and 1.77 rank these two floods as numbers 51 and 54, just outside the list of the highest 50. Even so, the probability of exceedance of each of these floods is only about 1 in 3,000.

The stability of the line defined by the points shown in Figure 2 is indicated by the fact that approximately the same line would be obtained had the eight 1972 peaks not been used in the analysis.

Table 1 gives information on the 50 most extreme peaks used in plotting Figure 2. The second highest peaks during the period of record at each of the 50 stream gauging

Table 1. Summary of USGS data used in defining probability distribution of extreme floods.

Order Number	Station Number	Drainage Area (sq mi)	$\frac{Q_{max}}{Q_{50}}$	S	Z_L	Year of Peak	State	Number of Annual Peaks	Plotting Probability (percent)	$R_{0.3}$
1	6-0785	258	7.89	0.161	5.57	1964	Mont.	24	0.00044	38.8
2	6-0920	317	9.89	0.202	4.93	1964	Mont.	33	0.00108	24.1
3	6-0925	133	8.39	0.205	4.50	1964	Mont.	17	0.00172	18.5
4	3-2720	275	5.06	0.182	3.86	1913	Ohio	54	0.00235	14.1
5	1-6435	62.8	6.26	0.225	3.55	1972	Md.	22	0.00299	10.2
6	5-0115	61.0	3.26	0.148	3.46	1964	Mont.	17	0.00363	10.1
7	5-0110	121	4.10	0.178	3.45	1964	Mont.	55		
8	5-0145	31.4	3.56	0.163	3.38	1964	Mont.	53	0.00490	9.66
9	1-1245	27.7	7.47	0.272	3.21	1955	Mass.	32		
10	1-5555	162	5.44	0.230	3.20	1972	Penn.	43	0.00618	9.14
11	5-0100	74.8	3.46	0.174	3.11	1964	Mont.	17		
12	1-5755	222	5.68	0.244	3.09	1972	Penn.	33		
13	6-7095	302	14.87	0.397	2.96	1965	Colo.	29		
14	2-1120	493	5.05	0.244	2.88	1940	N.C.	57		
15	1-5900	8.5	7.17	0.302	2.84	1944	Md.	34	0.00936	7.17
16	3-2630	1,155	3.48	0.195	2.77	1913	Ohio	54		
17	3-1610	207	4.33	0.230	2.77	1940	N.C.	43		
18	1-5860	56.6	4.61	0.246	2.70	1972	Md.	26		
19	1-4750	6.8	7.11	0.331	2.57	1940	N.J.	27		
20	4-2035	479	2.53	0.161	2.51	1928	Vt.	54	0.01254	5.97
21	5-3700	64.8	3.58	0.222	2.50	1942	Wisc.	25		
22	1-6610	10.7	5.05	0.290	2.42	1950	Md.	20		
23	2-1380	171	3.80	0.240	2.41	1940	N. C.	31		
24	1-4395	117	4.02	0.250	2.41	1955	Penn.	62		
25	3-4830	427	3.08	0.205	2.38	1940	Tenn.	28	0.01573	5.55
26	6-8769	6,770	6.54	0.343	2.38	1951	Kan.	61		
27	1-6425	82.3	3.30	0.227	2.28	1972	Md.	35		
28	1-2030	133	4.56	0.292	2.26	1955	Conn.	42		
29	2-1305	64.0	2.99	0.211	2.26	1945	S.C.	18		
30	2-0270	92.0	4.60	0.295	2.25	1969	Va.	32	0.01891	4.86
31	4-2860	397	2.96	0.210	2.24	1928	Vt.	56		
32	1-4115	113	4.09	0.282	2.17	1940	N.J.	35		
33	1-5765	324	3.91	0.273	2.17	1972	Penn.	44		
34	1-1240	157	4.92	0.320	2.16	1955	Conn.	41		
35	2-0275	48.0	4.43	0.302	2.14	1967	Va.	21	0.02205	4.55
36	2-0240	649	3.18	0.239	2.10	1969	Va.	32		
37	3-2740	3,639	2.75	0.214	2.05	1913	Ohio	55		
38	3-4810	42.0	3.08	0.254	1.93	1940	N.C.	21		
39	1-5560	291	2.62	0.217	1.93	1936	Penn.	54		
40	6-1640	2,299	3.09	0.256	1.91	1952	Mont.	55	0.02528	4.18
41	1-5730	333	2.76	0.231	1.91	1972	Penn.	54		
42	5-1010	162	4.69	0.353	1.90	1950	N.D.	30		
43	5-0130	238	2.26	0.201	1.88	1964	Mont.	43		
44	3-1625	277	3.18	0.267	1.88	1940	N. C.	40		
45	1-5835	59.8	3.66	0.303	1.86	1972	Md.	27	0.02847	3.90
46	1-1220	169	3.35	0.287	1.83	1938	Conn.	42		
47	6-0730	123	2.95	0.256	1.83	1964	Mont.	30		
48	1-6205	23.4	3.39	0.290	1.83	1949	Va	24		
49	5-3325	503	2.24	0.192	1.82	1941	Wisc.	54		
50	1-4825	14.6	4.52	0.363	1.80	1940	N.J.	28	0.03166	3.69

Notes: Q_{max}/Q_{50} is the ratio of the highest peak discharge during period of record to 50-year flood. $Q_{50} = F(Q_{50,UC}) + (1 - F)(ADJ)\,Q_{50,C}$, where $Q_{50,UC}$ is LPT3 curve for full sample, $Q_{50,C}$ is LPT3 curve for sample with highest peak removed, F is Kirby weight factor, and $\log ADJ = (1.56 - 0.8 \log N)\,F(LSD_{UC}) + (1 - F)\,LSD_C$, in which LSD_{UC} and LSD_C are the logarithmic standard deviations for uncensored and censored samples.

$R_{0.3}$ is magnitude on scale for S = 0.3 computed as $\log R_{0.3} = 0.3\,Z_L$.

stations given in this table all have Z_L values less than 1.8 and could thus be ignored in computing the plotting positions. If the analysis were to be continued to lower values of Z_L, however, the second highest and perhaps even the third highest peaks at each station would have to be considered. The results given in Table 1 are preliminary in that preliminary values of generalized logarithmic skew coefficients and of generalized logarithmic standard deviations were used in computing the generalized slope of the flood-frequency curves at the 50-year recurrence interval. Furthermore the adjustment for bias in 50-year peaks computed from censored samples should probably be modified. The preliminary results, however, serve to show how extreme flood events compare with flood-frequency distributions based on more ordinary flood experience.

In summary, I would say that, when an unusually high flood is included in an array of flood peaks, computation of the 50-year peak by the LPT3 procedure should use a generalized skew coefficient and that considerable weight should be given to the 50-year peak computed with the highest peak deleted from the array. Extension above the 50-year peak should be a straight line on log probability paper to about the 1,000-year probability level, but the slope should be based in part on the average slope at all stations in the area. Floods that are more than two standardized units above the 50-year peak are so rare that their exceedance probability (less than one in 4,000) is comparable to that for damage caused by other natural disasters such as earthquakes, landslides, and tornadoes. In other words, I think that flood risk at sites that experienced an unusually high flood in 1972 was not changed to any great extent by the occurrence of that flood. Rather, we should be more concerned with the true flood risk at all sites including the risk due to uncertainty in defining the 50-year peak.

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THE WORTH OF DATA IN HYDROLOGIC DESIGN

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Reviews of current methodologies for the determination of the worth of hydrologic data indicate that each method has certain shortcomings. The simulation approach requires information concerning the statistical properties of the data that are not usually known. The Bayesian approach often leads to mathematically intractable relations. The two approaches may be combined by defining the data worth through simulation under the condition of assumed statistical properties and releasing the conditioning by making use of the prior distributions of the unknown statistics obtained from the Bayesian approach. This combined approach circumvents the problems encountered when either the Bayesian or the simulation approach is used exclusively. An example illustrates the use of the combined approach in evaluation of the worth of flood data in the design of highway crossings.

•THE CONCEPT of the monetary worth of data is one basis for answering questions such as how many data are sufficient to make an economically optimal design decision. This concept is an aspect of hydrologic analysis that has only recently received direct attention (1, 2, 5). Because a methodology for the determination of hydrologic data worth has remained unstructured for so long, the designer in most cases uses available data to formulate his design without assessing the possibility that collection of additional data might lead to a better design in the long run. As more and more pressures are exerted on land and water resources by the expanding economies of the world, design decisions must be improved, and one way to do this is through the collection of optimal quantities of data on which the decisions will be based.

Data in general have value or worth only if they are used in a decision process. Hydrologic data are no exception. Anticipation of the uses of data is thus a prerequisite of any statement concerning their worth. Those data that are never used or are only used improperly in hydrologic design have a maximum worth of zero.

The randomness of streamflow data tends to cloud the definition of their worth. Because the data are governed by the laws of probability, only probabilistic inferences can be made concerning their reliability in the design process, and the true optimal design is never known. However, as more data are collected, the design based on the sample approaches the true optimum in a statistical sense. Each data point that is collected has a current worth equal to its degree of improvement of the design. Because the optimal design, which is the measure by which improvement is gauged, is unknown, only probabilistic statements can be made concerning data worth. The expected value of data worth is a parameter that can be used to define the optimal alternative between additional data collection and immediate design.

The worth of a data set may be defined as the benefits foregone if it were not available. In the context of design, this definition is interpreted as the difference in net benefits between designs including and omitting the data being evaluated. Dawdy, Kubik, and Close (2) have used the benefits-foregone definition in conjunction with synthetic hydrology to evaluate data worth in reservoir design. They synthesized a 500-year realization of streamflow from which the optimal storage capacity of the reservoir was determined. The 500-year record was then partitioned into fifty 10-year records, twenty 25-year records, ten 50-year records, and five 100-year records. Reservoir capacities were designed for each record, and net benefits based on the 500-year record were computed for each design. Average net benefits for each class of record lengths were computed. The differences in average net benefits between two successive classes of record length were computed and equated with the expected value of the increment of data between the pairs of classes. Dawdy and others found that the unit worth of data decreases with increasing record length; i.e., the marginal value of data decreases with an increase in the level of information gained. Similar findings were reported by Tschannerl (5).

The simulation method just described can be adapted to any hydrologic design problem by the use of pertinent design procedures and the related cost and benefit functions. It does, however, have one serious restriction: The statistical moments of the distribution of the hydrologic variables must be known prior to the analysis. The method is thus limited to either an after-the-fact analysis, when sufficient data have already been collected, or a conditional analysis with the understanding that the answers have only a certain probability of being correct.

Davis (1) developed a methodology, based on decision theory and subjective probabilities, that avoids the inherent restriction of the simulation approach. The subjective approach is statistically elegant, but it often requires very complex mathematics in order to arrive at an answer. It treats the unknown population parameters as random variables with probability distributions defined by prior knowledge and available data. From these probability distributions a design is formulated that takes into account the randomness of the unknown parameters. The design and prior distributions are used to estimate the expected opportunity loss that is caused by imperfect knowledge about the parameters. The opportunity loss, which is also an unknown that is treated as a random variable, is the equivalent of the worth of an added record of infinite length. The prior distributions are next used to evaluate the expected value of the expected opportunity loss after the addition of one more unit of data. The difference between this expected expected opportunity loss and the expected opportunity loss is the expected worth of the added unit of data. The subjective approach thus estimates data worth for a single step into the future. Attempts to extend the method to multiple steps are thwarted by the complexity of the mathematics, which increases in degree equal to the number of steps considered.

Each of the methodologies thus have their benefits and their deficiencies. Because none of the deficiencies mentioned above is common to both, it is possible to combine the two methodologies and arrive at a blend that is superior to either used alone. This is accomplished by defining the expected values of data worth under the condition of known statistical parameters by the simulation method. The conditioning is then released by use of the prior distribution of the parameters, which is derived from the subjective approach. This procedure is illustrated in the following hypothetical example.

Suppose that a new highway is to be constructed between two towns in western Oregon and that this highway must cross 10 water courses that have basins upstream of the proposed crossings that are described by the information given in Table 1. The engineer's design manual tells him to design the crossing structure so that it will pass the discharge of the 50-year flood without incurring any damage. There are no discharge records on any of the streams, so the engineer decides to use regression estimates (4) of the 50-year floods as his design discharges. The regression estimates, of course, contain errors, in this case a 40 percent standard error of estimate; and the resulting designs will be suboptional. The suboptional designs result in benefits foregone in the form of added construction costs for overdesign and added maintenance and replacement costs in the case of underdesign. These benefits foregone are the potential worth of the uncollected data.

Assume that the present value at some specified discount rate of benefits foregone at each crossing is related to the difference between the logarithms of the design discharge Q_D and the true 50-year flood Q_{50} by the function shown in Figure 1, which could be defined by simulation of long flood records and by computing the resulting costs. The present cost C_D of the structure if Q_D were an optimal design is related to the design discharge in the manner shown in Figure 2.

Implicit in the logarithmic regression analysis that was used to define the value of Q_D is the assumption that the residuals or in this case the errors of the logs of the dependent variables are distributed normally about the logs of the regression estimates.

Under such an assumption, Hardison (3) has provided the means for converting from standard error of estimate in log units to equivalent years of record; i.e., a certain standard error from the regression analysis can be expressed as the number of years of actual data that would have to be analyzed to give an estimate of the parameter that had an accuracy equal to that of the regression estimate. Making use of this analogy reduces the simulation phase to the definition of Figure 1.

The assumption of log normality for the residuals from the regression designates that the log-normal distribution should be the family of the prior distribution of unknown true values of Q_{50}. The prior distribution of the logarithms of Q_{50} would thus be normal with a mean of log Q_D and a standard deviation equal to the standard error of estimate of the regression analysis. The standard error of estimate rather than the standard error of prediction is used only for computational convenience and for demonstration purposes.

The expected opportunity loss can now be computed for each of the crossings by the integral equation

$$XOL(i) = \int_0^{\infty} P_i(Q_{50}, Q_D)\, f_i(Q_{50})\, d\,Q_{50} \tag{1}$$

where

$XOL(i)$ = expected opportunity loss at the ith crossing,
$P_i(Q_{50}, Q_D)$ = penalty function or benefits foregone for a design at the ith crossing based on Q_D when the true value is Q_{50}, and
$f_i(Q_{50})$ = probability density function of the prior distribution of Q_{50}.

Transforming the values of Q_{50} and Q_D to their logarithms and taking note of the form of the penalty function

$$P_i(Q_{50}, Q_D) = C_D(i)\, p(\log Q_D - \log Q_{50}) \tag{2}$$

expand Eq. 1 to

$$XOL(i) = C_D(i) \int_{-\infty}^{\infty} p(\log Q_D - \log Q_{50})\, f_N(\log Q_D - \log Q_{50})\, d(\log Q_{50}) \tag{3}$$

where

$p(\log Q_D - \log Q_{50})$ = function of Figure 1 and
$f_N(\log Q_D - \log Q_{50})$ = normal prior distribution of log Q_D.

The integral of Eq. 3 is independent of the value of log Q_D because the distribution of log Q_{50} is always symmetrically distributed about it. The value of the integral is thus a function only of the standard error of estimate of log Q_{50}. Equation 1 can be further simplified

$$XOL(i) = C_D(i)\, I(s) \tag{4}$$

where I(s) = integral of Eq. 3, which depends on s, the standard error of log Q_{50}. The relation of I(s) to s is shown in Figure 3 along with the equivalent lengths of record.

The total expected opportunity loss for all 10 sites is the sum of the individual values. Because the integral of Eq. 3 has a common value at each crossing, the total expected opportunity loss is expressed as

$$XOL^{*} = I(s) \sum_{i=1}^{10} C_D(i) \tag{5}$$

Table 1. Basin parameters, design discharges, and theoretical costs for stream crossings.

Station	Drainage Area (square miles)	Storage (percent + 1)	Elevation (10^3 ft)	Annual Precipitation (in.)	Temperature Index (deg F)	Soils Index[a]	Q_D[b] (cfs)	C_D (thousands of dollars)
1	22.3	1.67	2.44	96	29	5.6	3,830	49.0
2	52.7	1.06	4.09	58	24	5.6	4,600	56.0
3	73.0	3.56	2.99	120	29	5.6	13,400	113.0
4	107	2.77	2.62	115	29	5.6	17,600	136.0
5	113	2.15	4.46	60	21	5.5	5,750	64.0
6	117	1.51	4.14	62	24	5.0	9,870	92.0
7	246	5.23	3.76	58	25	4.6	13,200	112.0
8	258	1.18	4.38	60	24	5.4	20,900	152.0
9	392	1.34	4.08	58	24	5.2	27,300	182.0
10	924	2.38	3.97	58	25	4.9	56,000	294.0
Total								1,250.0

[a]Compiled from U.S. Soil Conservation Service maps.
[b]$Q_D = 1.70 \times 10^{-4} A^{0.903} S_t^{-0.310} E^{0.741} P^{1.94} T_f^{2.714} S_i^{-0.660}$.

Figure 1. Penalty function for inaccuracies in the 50-year flood.

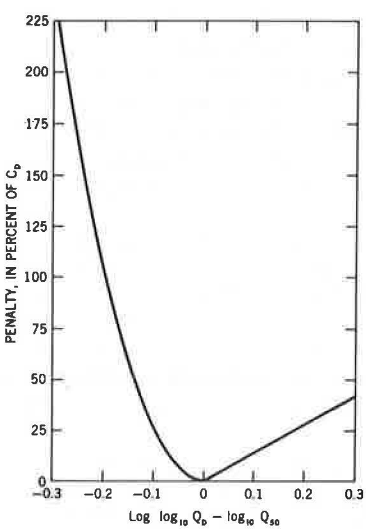


Figure 2. Cost function for optimal design.

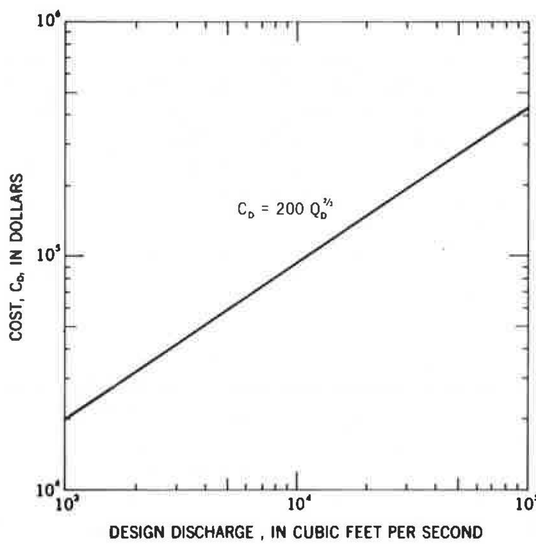
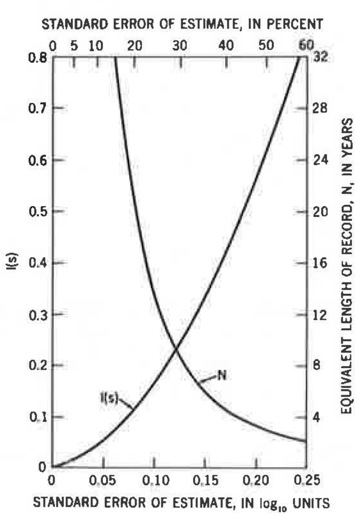


Figure 3. Equivalent length of record and integral from Eq. 3 as functions of standard error of estimate.



The regression analysis of Lystrom (4) yields a standard error of estimate for the 50-year flood of 40 percent, which is equivalent to 0.17 log units. The regression estimates of Q_{50} that were derived from the basin parameters of Table 1 are also given in that table. If these data are used in conjunction with Figures 2 and 3, Eq. 5 can be evaluated for the example. The expected opportunity loss is $524,000, which is the expected present worth of perfect knowledge concerning values of Q_{50} at each crossing.

Because perfect knowledge is an unattainable state, the expected opportunity loss is only a gauge by which the worth of additional data can be measured. The expected change in expected opportunity loss at any point in time of a data collection program is the expected worth of the additional data that might be added to that already collected up to that time. For instance, if a data program that provided the equivalent of 10 years of data were instigated, the 50-year flood would have an accuracy of 28 percent, according to Lystrom (4); a 25-year equivalent record would have an accuracy of 16 percent. Using these two figures in the analysis gives expected expected opportunity losses of $286,000 and $115,000 for the 10- and 25-year records. When each of these expected expected opportunity losses is subtracted from the expected opportunity loss of the previous paragraph, the expected worth of the 10-year records is found to be $238,000, and the 25-year records are found to be worth $409,000. The difference between these two values, $171,000, is the added worth of extending the records from 10 to 25 years.

The above analysis can be repeated for each consecutive year of data added beyond the currently available 5-year equivalent. By the use of 1-year increments the marginal worth of each added year of record can be computed. Comparison of the marginal worth with the cost of obtaining the data indicates the optimum record length or its equivalent that should be collected for the design. Data should be collected as long as their expected marginal worth exceeds their expected marginal cost. The expected marginal costs must include the costs of benefits foregone by delaying the design to obtain the added data.

It would be very rare, indeed, if the value of additional hydrologic data were sufficient to delay the construction of a highway. However, because the design floods contain certain unknown errors, it may be economically efficient to consider the collection of data in order to improve the estimates for those structures that will have to be replaced in the future. If the designer decides to make an investment in data, he has at least two alternatives at his disposal. He can invest in a gauging program at the present crossing sites, or he can help finance improvement of the regression analysis, thereby making better estimates available in the future.

The at-site gauging program could be accomplished by the use of relatively economical crest-stage gauges; but, because it is not known at which sites the information will be needed, the program might result in a gauge at each site. This in not necessarily the case, however, because there usually is some transferability contained in the information gathered at nearby sites. If the transferability is sufficient, a less-than-saturation gauging program will be optimal.

Improvement of the regression estimates is probably the most effective means of investment. It not only yields better estimates at the sites in question but also improves those throughout the region in which the regression analysis is valid. Therefore, other suboptimal crossing structures in the region may be improved as they are replaced, and new structures will also benefit from the added level of regional information.

Improvement of the current version of the regression model may be accomplished by reducing any one of the three types of errors that are contained therein: time-sampling, space-sampling, or modeling errors. Reduction in time-sampling errors is the result of extending the streamflow records at the sites that were used in the current analysis, thereby improving the estimates of the streamflow statistics, in this example Q_{50}, that are used as dependent variables.

Decreased space-sampling error is effected by the initiation of new gauging stations at sites that will provide a broader range of variables for the analysis. The new stations should not be included in the analysis, however, until their time-sampling errors are small enough so that they will not add more noise than information. In the current example, installation of gauges at one or more of the proposed crossings would serve the double purpose of reducing space-sampling error in the regression analysis and providing at-site records for the chosen crossings.

Modeling errors can be reduced only by improving the structure of the model. This may often be done by including additional independent variables, in which case data describing the new variables are required.

At the present time it is not possible to state which source of error should be attacked first. However, the economic efficiencies of data collection as a means of reduction of space- and time-sampling error are being investigated, while further developments in the understanding of the physical processes of hydrology indicate that improvements in the structure of the regression models also are forthcoming.

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DISCUSSION

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Matters of considerable practical importance are dealt with in the paper by Moss and Dawdy. In particular, the basic concept that the worth of hydrologic data should be expressed in design-directed economic terms has not received adequate attention in the past, and this concept is well worth stressing. The point was expressed succinctly by Linsley (6) in 1965: "The collection of data is not an end in itself. Data are collected for use and the requirements of this use should govern network design."

Not long afterward, the Canadian Department of Energy Mines and Resources (now the Department of the Environment) commissioned various studies directed, in part, toward developing a use-oriented, cost-benefit approach to hydrometric network planning. Techniques developed in Canada, largely as a result of these studies, have recently been given a broader audience through papers presented in the United States. In several of these papers (7, 8, 10) it has been remarked that on a regional basis the economic loss resulting from uncertainties in hydrologic data varies approximately as the square of the relative uncertainty.

In this regard, it is interesting to note that the same parabolic relation between relative economic loss and relative uncertainty can be shown to apply to the results of Moss and Dawdy. Their function I(s) can be taken as the relative loss (Eq. 5), and the standard error of estimate, in percent (the upper abscissa of Fig. 3, which is assigned the symbol S_r for the purposes of this discussion), can be taken as the relative uncertainty. The parabolic approximation $I(s) = KS_r^2$, with a K-value of about 2.7×10^{-4}, provides a reasonable approximation to the curve labeled I(s) in Figure 3.

It may also be remarked that in the paper by Moss and Dawdy the method of calculating expected opportunity loss would appear to depend on a preexisting knowledge of the optimal return period as given by "the engineer's design manual" or the subsequent assumption by which the "true 50-year flood" is taken as a datum. However, if one wishes to obtain a scientific basis for the design of hydraulic structures, the optimal return period must itself be taken as a dependent variable. In a previous paper concerned with design strategy for small flood-control structures (9), we showed that the optimal return period would differ according to whether the "perfect knowledge" or "real-world" case was being considered. In our paper, an equation was first obtained that expresses the perfect knowledge return period in terms of factors such as interest rate, damage ratio (cost of typical damage event/cost of optimum structure), and flood-frequency exponent. This case is denoted as perfect knowledge because it assumes that the true values of the various factors are known. The analysis was then extended to the real-world case where uncertainties in the component factors combine to give an overall uncertainty in estimating the optimum. It was found that the penalty resulting from underdesign caused by this uncertainty is greater than that for an equivalent overdesign, and hence the real-world optimum should be larger than the estimated perfect knowledge value.

The example given in Moss and Dawdy's paper will also impose a larger penalty for underdesign, as indicated by the relatively greater steepness of the left limb of Figure 1. On the basis of our paper (9), it is suggested that in this case the authors have used a perfect knowledge design strategy for a real-world case. If the real-world strategy had been used, the design discharge would have been progressively increased with increasing uncertainty. This apparent overdesign ensures that the probability of occurrence of very large damage (points far to the left on Fig. 1) remains reasonably small. It follows that the use of this real-world design strategy will produce a smaller estimated opportunity loss than the strategy adopted by Moss and Dawdy.

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Moss and Dawdy have presented an approach to evaluating the worth of data in hydrologic design that is generally appropriate. As is usually the case in such studies, the difficulty lies in applying the conceptual approach to a real problem. An additional difficulty is that the person appraising the worth of data is rarely in a position to influence data collection programs in any significant way.

One example illustrates the difficulty in applying the approach to a real situation. The authors use a regression analysis that yields values of Q_{50}. Then the expected opportunity loss is calculated. It is difficult to decide whether such a calculation is better than no calculation at all, because the losses that would occur from floods whose peak discharge exceeds Q_{50} depend more on the hydrograph than on the peak discharge. A stream crossing, bridge or culvert, does not automatically fail when a given peak discharge occurs. Ponding, duration of discharge above that peak discharge, and site characteristics must be considered. This does not imply that regression equations have no place in design. They may provide a quick and adequate means of sizing a waterway opening. A full analysis that included consideration of the full hydrograph would yield a better design, from an economic standpoint, but not necessarily a safer design.

Data collection programs are in need of optimization, especially in the area of coordinating the collection of precipitation and runoff data, perhaps our greatest weakness in data collection at the present time. The worth of the data should have some bearing on decisions to optimize those data collection programs. However, it still seems that our larger needs are to find logical, defensible methods of using the data we have in design of hydraulic structures. This would allow a much better appraisal of the worth of the data. Then, the approach suggested by the authors (which may be ahead of its time) can be fruitfully used.

AUTHORS' CLOSURE

We appreciate the discussions of our paper by Wilson and Watt and Willeke. In general we agree with their comments, which may be considered as extensions of our ideas. Wilson and Watt point out that the loss function relation is quadratic. This was a basic assumption, rather than a conclusion, because a quadratic average loss function was considered as relatively realistic. This can in no way, however, then be used to justify that assumption. We agree that the use of a 50-year design flood was an oversimplification. The design discharge itself should be a decision variable, but we felt that explicit consideration of that problem would have obscured the major point, which was that data networks should be judged in economic terms. The uncertainty in knowledge of the proper design flow affects the results of our study, just as Wilson and Watt outline, by causing hedging against underdesign. Introducing the design discharge as a decision variable will reduce the estimated expected opportunity loss (11).

As stated earlier, the loss function is an assumption and an oversimplification of the facts. As Willeke pointed out, it may be unrealistic. However, it is a first step. We should not refuse to walk merely because we cannot win an Olympic Gold Medal. More realistic loss functions will lead to better decisions. Until they are developed, any "optimization" in fact is suboptimal. We hope we can start to converge toward more defensible and, perhaps, better information networks.

REFERENCE

11. Close, E. R., Beard, L. R., and Dawdy, D. R. An Objective Determination of Safety Factor in Reservoir Design. Jour. Hydraulics Div., Proc. American Society of Civil Engineers, No. HY5, May 1970, pp. 1167-1177.

DESIGN CRITERIA AND RESEARCH NEEDS

Vernon Hagen, Hydrologic Engineering Branch, U. S. Army Corps of Engineers

•THE OCCURRENCE of an extreme flood event, such as Hurricane Agnes, requires all those involved in flood prevention activities to review procedures and criteria currently in effect.

The degree of protection provided by a flood control project refers to that amount of the flood that can be controlled to essentially nondamaging effects. This can be accomplished by a reservoir reducing the peak flow or by local protection works such as a levee, channel improvement, flood wall, diversion, or combinations of more than one type of work.

One might wonder what the degree of flood protection provided by Corps of Engineers projects has to do with highway design. Well, I am sure most people involved in highway work realize that one of the most critical problems involved in flood protection work is the relocation of highways and streets. The costs associated with raising or replacing bridges, protecting embankments, gating culverts, and other necessary highway costs are often a major portion of the total cost of a flood protection project.

In selecting the degree of protection provided by a project, Congress has generally insisted that the average annual tangible benefits exceed the average annual costs of the project. There are very few exceptions where intangible benefits have been great enough to convince Congress that a benefit-cost ratio less than unity should be permitted. It therefore follows that a major constraint in project formulation is that the benefit-cost ratio be equal to or greater than one.

Because there are not nearly enough federal funds to provide everyone flood protection, it is important that available funds be used where the greatest return can be realized. This return can of course be in the form of either tangible or intangible benefits. In the formulation of Corps flood control projects, it is required that the point of maximum net tangible benefits be established. This is determined by adding increments of flood protection until the benefits of the increment are exactly equal to its costs. The point of optimum economic return thus established is the beginning level for selecting the degree of protection. When intangible benefits are great enough, the degree of protection may be increased to a large flood such as the standard project flood, provided the benefit-cost ratio will permit. In some agricultural areas, intangible benefits are relatively small, and the degree of protection is then usually limited to the economic optimum. There are many items that enter into the intangible considerations when one evaluates how far beyond economic optimum he should go in selecting the degree of protection. One of the more important items is the potential for loss of life when the design flood is exceeded. This potential is related to factors such as the rate of rise in flood levels, type of project design, warning facilities, escape routes, and many others. There is not enough time to go into detailed discussion of project formulation; however, it should not be a casual undertaking.

Should the occurrence of Hurricane Agnes and other severe storms like the Rapid City disaster alter the basic criteria used by the Corps in selecting the degree of protection to be provided by flood control projects? Personally, I do not believe they should. I think the criteria are basically good; however, their application could stand some improvement.

The experience of great damage and loss of life by unusual floods provides a vivid example of the need to consider intangible benefits in project design. Immediately following large floods, local interests have a keen desire to provide a high degree of flood protection or control on development in floodplains. As time passes and memory dims, they become more concerned with project costs or earnings to be gained by intensive use of the floodplains. Lessons learned from the large floods should be used to adhere more closely to prudent design criteria and should discourage those primarily concerned with maximum economic gain or minimum expenditure.

I have indicated some of my views on formulating the degree of flood protection to be provided and the effect of Agnes on this formulation activity. Now, I would like to comment on the effect of Agnes on some of those hydrologic tools we use in project formulation.

Two hypothetical floods that are of considerable significance to us are called the probable maximum flood and the standard project flood. The probable maximum flood is used to size spillways and heights of major dams built by the Corps. It is defined as the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in a region. The standard project flood is generally used as the upper level of flood protection to be made available by construction works. Its definition is essentially the same as the probable maximum flood except that we substitute the words "reasonably characteristic" in lieu of "reasonably possible" and add a phrase that excludes extremely rare combinations. The standard project flood is usually from 40 to 60 percent of the probable maximum flood in the eastern two-thirds of the United States. The storm rainfall values selected to derive a standard project flood have generally been exceeded by 10 to 15 percent of actual storms experienced within the region.

The hydrometeorological branch of the Weather Service and hydrologic engineers in the Corps have been examining the rainfall characteristics of Agnes. Although results are not complete, initial studies indicate that any changes in probable maximum precipitation or in the generalized criteria for standard project storms would not be appreciable. Agnes was a really extreme storm for some locations and for large areas; however, it did not exceed probable maximum precipitation criteria for the area it covered. There were a few areas where the rainfall exceeded standard project storm values, but this was expected to occur on rare occasions.

In the reevaluation of the Susquehanna River hydrology, the Hydrologic Engineering Center in Davis, California, found that many of the routing characteristics of the basin had to be revised from previous assumptions to reproduce such an extreme flood as Agnes. These changes, of course, resulted in new estimates of standard project flood peaks at some locations. Revisions in peak flow-probability relations have also been developed since Agnes. Although the peak flows of Agnes provided the extreme events at many gauging stations, they were only a partial reason for major changes in frequency curves. Many of the older curves had been developed with less data and were derived by superseded procedures. The recent studies by the Hydrologic Engineering Center involved regional frequency analysis, which should provide better results than single-station studies.

In summary, I do not contemplate any immediate major changes in the Corps's design criteria or hydrologic procedures because of Agnes. However, I anticipate more interest in intangible benefits when project design is formulated in the future.

Research such as that being conducted by Hardison, Reich, and others may provide us with new techniques for improving our hydrologic tools. In addition to flood probability procedures, another important area of hydrologic research is the development of conceptual models of river basins that permit optimum management of reservoir systems and accurate runoff forecasting.

Walter Hofmann, U. S. Geological Survey

•THE Geological Survey does not have construction responsibilities, so I cannot comment too much on design criteria. I would like to make the point, however, that we should continue to strive for improvement in the flood-frequency analysis techniques, and, what is more important, we need to achieve a better degree of uniformity or agreement on the results from the flood-frequency analyses. It behooves all of us in the federal establishment to try to get together and come up with an answer that will eliminate some of the confusion in flood-frequency analyses that results from different magnitudes of floods being assigned the same frequency. Speaking for the Water Resources Division of the Survey, we are attempting to improve what we do in terms of data collection activities. We are coordinating our activities with the National Weather Service in an attempt to improve not only the data collection side but also the forecasting activities by input from both agencies. This coordination is still in the formative stages, but I am sure improvement will result in both activities.

The last point I wanted to make was in connection with the report on bridge and culvert failures by O'Donnell. Carl Nordin of our Fort Collins office attended the meeting of the Surface Water Drainage Committee in August, and I think the question of bridge failure was raised. At that time, he got the impression that very little would be done to document the failures occurring as the result of the 1972 floods. Apparently, though, there has been considerable documentation. The point I wanted to make is that it would be very helpful if the statistics of bridge failures could be collected on a nationwide basis in a uniform format so that the current design practice could be evaluated and the relative importance of additional research on scour problems, approach design, structural design, bank protection, and the like could be identified. Nordin has suggested that the Geological Survey take the lead in a project to develop this information in cooperation with the state highway departments. If funding permits in fiscal year 1974, we will try to make a start on this, but it will be a bigger job than we can handle with our resources alone. Perhaps it is something that this Surface Water Drainage Committee could take the lead in. Nordin recommended that, for selected floods, the number of bridge failures be determined and classified according to type of failure, e.g., pier scour, approach failures, abutment failures, structural failures from debris or ice buildup, some combination of these, or any other factors deemed significant. There are other statistics that could be compiled at the same time, and we would be glad to work with the Surface Water Drainage Committee to develop the questionnaire.

Frank L. Johnson, Hydraulics Branch, Federal Highway Administration

•WE HAVE HEARD about the 50-year storm quite a bit, and I assume that it is known that we generally design for the 50-year storm. But there is more to it than just that, so I want to quote a portion of an FHWA policy memorandum that most highway departments follow in designing structures.

> All culverts and bridges over streams shall be designed to accommodate floods at least as great as that for a 50-year frequency or the greatest flood of record, whichever is greater . . . with backwater limited to an amount which will not result in damage to upstream property or to the highway . . . . For highways other than those on the Interstate System, the design flood may be less than a 50-year frequency where conditions warrant lower standards. The flood frequency selected for design should be consistent with the magnitude of damage to adjacent property and the importance of the highway.

The word "accommodate" is intended to imply that the design flow will not necessarily pass through the structure. The significance of this is that an underestimate of the design flood does not necessarily mean that the structure will fail if a flood in excess of the estimated flood occurs. Because of the recent rare flood events, and for other reasons, we are currently reexamining these design criteria. In April, the Water Resources Council issued a bulletin, Flood Hazard Evaluation Guidelines for Federal Executive Agencies. It was issued to implement provisions of Executive Order 11296, which requires that to preclude uneconomic, hazardous, or unnecessary use of floodplains requires that the flood hazard be evaluated in the planning of federal or federally aided facilities. In these guidelines, two approaches are identified for purposes of flood hazard evaluation: the plan approach, under which flood hazard areas are delineated and long-range land use regulations are established, and the case approach, under which decisions regarding floodplain usage are made case by case. The case approach is the one that will generally apply to highway designs. These guidelines indicate that the 100-year flood should be used as the basic flood in evaluating the flood hazard and that any encroachment on the floodplain will permit conveyance of the basic flood without increasing flood heights or velocities to an extent that will cause significant damage to existing or anticipated upstream or downstream development. So, we need to look at the policy under which we are operating to see whether the basic flood concept creates a need for changes in our design criteria.

The executive order goes on to state in another guideline that floods greater or less than the basic flood may be used in the evaluation, as appropriate. So, as a result of the rare events experienced recently and these new guidelines, we have been looking at the policy that I quoted earlier. Tentatively, we have come up with something that is slightly different, not a great deal, but somewhat different, that we think will meet the objectives of the guidelines and comply with the executive order as well as take care of us in the rare flood events. Here is the way it currently reads. (The statement is still in the formative stage and is subject to change.)

> All designs for highways that encroach on flood plains shall permit conveyance of the 100-year flood without significant damage to the highway or other property. All culverts and bridges shall be designed to pass at least the 50-year flood without traffic interruption, except that structures for low traffic volume highways may be designed for recurrence intervals compatible with traffic volumes, construction costs, risk of flood related accidents and costs to repair probable highway damages from floods larger than the design flood.

This is intended to recognize that flood events in excess of the design flood will occur somewhere each year. It is not possible, because of money constraints, to build all structures to pass the biggest flood that might happen in that location. So, the only answer appears to be to build highways so that extensive damage will not occur, either to the highway or to other property. A highway fill can be replaced in short order and at a relatively small cost compared to the time and cost involved in replacing a bridge. The key words in the new policy, as written, were borrowed from the guidelines for flood hazard evaluation: "Any encroachment on the floodplain will permit conveyance of the basic flood." Highways that encroach on floodplains can and should be designed to permit conveyance of floods that exceed the design flood.

Samuel V. Fox, Texas Highway Department

•THERE IS an old advertising adage that goes something like this: "Tell them what you're going to tell them; tell them; and then tell them what you told them," and, by now, if you have not been told about catastrophic floods, there is not much I am going to be able to add. On page 3, Thomas presents maps that show no floods in Texas for 1972. He explained that there were floods that he could not show because time limited taking into account all the significant floods that had occurred in 1972. Herr alluded to the fact that it was difficult to get information out of Texas.

I am afraid that probably our simple way of life sometimes prevents people in the outside world from really knowing what goes on down in Texas, but I want to take this opportunity to quietly report something to you. I think we've done it again! I believe we have set another record in Texas! (You would be disappointed if I did not say that.) We had a "little" flood that occurred on May 11 and 12, 1972, in the Guadalupe River basin between San Antonio and Austin that evidently got covered up. We had rainfall intensities that ranged to more than 10 in. in 1½ hours. There were 16 people killed, and the event possibly yielded a record unit runoff. I know it set a new record in Texas, and how it compares to the record for the rest of the United States I am not sure. From a drainage area of 0.38 square mile, we got a runoff of 2,510 cfs, which translates to a unit discharge of 5,236 cfsm. Please note that we did not lose a single highway or a single bridge. It happened at approximately 11 p.m., and many of the people that were killed were asleep in their homes. For the most part it was just simply a wall of water.

We have been concerned about the catastrophic flood, also referred to as the super flood by the AASHO Task Force on Hydrology and Hydraulics. Those of us on the task force have talked about what kind of guidelines AASHO should include in telling the state hydraulic engineers just what they should do in evaluating these so-called super floods, maximum events, catastrophic occurrences, or whatever you wish to nickname them. Each highway engineer, if he has been doing his job, has also been concerned. This is nothing new; it just seems to be brought to mind every time one of the real big ones, like Agnes, occurs. But somewhere, for instance in Texas, every single year there is at least one and sometimes several floods that exceed the 50-year flood by more than 2, 3, or 4 times. We have tried, in the past, to avoid analyzing the big flood. Now our thinking is to look the big floods over, to analyze them, and to try to determine, without any thought of really designing for that particular flood occurrence again, what contributions or feedback we can get from a flood that might affect our normal design procedures within the realm of economy.

It has just been in recent years that we have really begun to properly document the big floods. Other authors in this Record have talked about documentation and have shown some excellent documentation. To me it is most important to get out while the flood is occurring, if possible, or very soon after the flood to find out what happened with regard to the highway and the highway structures.

The highway departments now must take a different tack. In dealing with these flood hazards, we have to begin at the time the flood is going on and, immediately after, to listen to what the local citizens have to say to us. They will be in a state of shock; many of them will have just lost their homes; some of them will have just lost their loved ones; they will not have money to fund remedial Corps of Engineer projects. Certainly a lot of the counties do not have enough money to do much good. Emergency relief is not available right there on the spot. The man they can look to is the "ole boy" that holds the highway purse strings, and I am not saying that we have all the money; we just happen to be the men on the job in many of these floods, and there are very few floods that do not involve highways in one form or another. We have to listen to the citizens and try to calm their fears by explaining the effects of highway involvement and our interest in doing what we can and not just tell them this is an occurrence we cannot design for.

Sandahl mentioned blowing the highway to relieve a flooding condition. All of us have thought about this from time to time. Sandahl spoke of blowing the highway if warranted after studying what effects it might have downstream and upstream. I would hope that in all cases of this kind a hydraulic engineer would be involved in the decision. All available expertise should be utilized.

We have lost highways and structures in Texas, and evaluation of these losses points toward two basic causes: drift and scour. We have lost structures from drift in two different ways—one from buildup that blocks the bridge and causes the water to increase in head to the point where it produces a very erosive velocity under the bridge, taking out the foundations. We have also had failures, pretty dramatic failures, from the impact on structures of large drift being carried down by the currents. Scour has taken out roadway embankments, header banks, and bridge foundations.

We are not changing our design criteria to any great extent, and I can say without hesitation that we are not designing the opening to accommodate the catastrophic flood any more now than we ever did. Design, of course, varies depending on the criteria established to define the design boundaries or limits. We are not designing for the catastrophic flood because we do not know what the magnitude of the catastrophic event is going to be. Is the catastrophic flood going to be two times a 50-year flood, five times a 50-year flood, or, as mentioned by other authors in the case of Hurricane Agnes, 62 times a 50-year flood? Hardison said that our goal should be to do the best we can to determine the 50-year flood and its true risk values and, then going from there, to drop the big flood magnitudes as outliers. The 50-year flood represents a sound design boundary because we feel its magnitude is predictable and design results are within the realm of economy.

So, although we have not changed our basic waterway design, we are, through the philosophical approach, realizing that we are encroaching on an existing system when we build a highway across or adjacent to a floodplain and, consequently, must be aware of the effects we impose on that existing system. This seems to be an important message of Executive Order 11296 to which Frank Johnson referred earlier.

One consideration concerning the catastrophic event is the availability of adequate alternate routes. In coastal areas, entire cities can become isolated when all highways serving the area are inundated and impassable. We should not overlook the possibility of providing, where feasible, accessible routes.

All water during the extreme flood does not have to go through the highway structure. We consider relief grades in roadway approaches to bridges that would allow the water, if dammed up at the bridge by drift, to have some type of relief by allowing overtopping of the roadway grade. We also consider raising the low point of the superstructure of the roadway bridge to clear the known high-water datum that has occurred in the past.

We also are becoming more conscious of the classes of dams and levees that are upstream and downstream from structures and what would happen if they failed. Who designed them and how valid the design is become important. Are they flood control structures, water supply structures, or just what?

We need to use in all cases, at least in my opinion, flow-through railing on all hydraulic bridges. This will allow better passage of some drift. Although it is far from a cure, it is better than building a parapet wall of concrete or creating something that will cause the water to run off around the ear wall or wingwall and take out approach slabs and put extreme pressures on the bridge. These are the times when the impact loads from fairly heavy drift have done the most damage.

We need to consider the foundations. I believe it is in South Dakota that they have shied away from the spread footing and now have gone to piling and drill shaft. This is a simple move but nevertheless a move worth taking.

We need to constantly update motorist warning systems. Maintenance people are well aware of what needs to be done when one of these floods occurs to adequately warn the motorist.

That hurriedly covers our current thinking. It is more philosophical than it is an absolute design approach. The research that might be suggested from studying the catastrophic floods is mostly under way and involves two areas of concern: hydrology and risk. The state hydraulic engineer, who is not a hydrologist, has to define, when using something like the Log-Pearson Type III analysis, what to do with outliers that represent catastrophic storms. I was glad to hear Hardison's presentation because he has possibly furnished us a tool to deal with outliers that I am going to study more closely. One thing we need is some way to evaluate whether we throw the outliers out or leave them in for our design.

The second item involving research is risk evaluation, and a pioneer study by Ken Young of Water Resources Engineering, Inc., in cooperation with FHWA, is already under way. They are seeking a better method for evaluating risk, which is really, when we get down to it, the name of the whole game. We have not been losing modern structures built in the last 15 years to the catastrophic floods. Maybe we are just fortunate. I think most of it relates to the fact that we are designing our foundations better. Our structure openings and grade lines are better designed because of the use of better hydraulic and hydrological tools. The bridges that have been lost in the last 15 years have been older structures, some on spread footing and some on timber pile. So we believe, all in all, the highways we are building today are standing the tests of these large unusual events very well. Now all we have to do is just fit our design philosophy into the whole system of things, and I think we will be on the road to success.

Brian M. Reich, Pennsylvania State University

•THE ONE THING about being the last man is that most things worth saying have already been said. Perhaps returning to the actual title of this session will allow us to come up with something on current design criteria and research needs. So I want to talk mainly about building a bridge between research and methods that can be applied by designers. There is an information gap, extension gap as our agricultural friends call it, between the results of research that has been done in various agencies or at universities and methods we see applied in engineering practice. It is quite understandable that there should be some sort of a gap here. There should be a providing period of these new ideas. We should not just pick up every new idea and run with it; so we expect a certain amount of this information gap. There are, however, some specific things that I think have gone on a little too long, which are particularly pertinent in designing highway bridges.

First, let us address the topic of risk, which has been mentioned often today. I am glad to see that almost every one of the panelists discussed the needs for more research, particularly research in flood-frequency analysis. There are certainly quite a lot of people studying this subject today. Figure 1 shows a flood-frequency curve of the Susquehanna River at Harrisburg. The dots represent the observed floods, the highest one, of course, being the 1972 Agnes flood. The second largest is the 1936 flood, which was previously considered the record flood. The series of annual maximum river records comprises the 82 dots. We consider ourselves extremely lucky if a river record is so long. Various curves have been mathematically fitted. The dotted straight line is the Gumbel flood-frequency curve, classically mentioned in introductory textbooks. The Log-Pearson Type III, which was proposed by the Water Resources Council, is shown with the calculated coefficient of skewness of the logs, CSL = 0.670. Suggestions have been made by some to regionalize the skew or arbitrarily take it as zero and to recompute the Log-Pearson Type III curve. The data can be fitted rather well with the Log Gumbel Curve, which simply requires taking logs of the floods before applying the Gumbel technique. This approach, also known as the Frechet distribution, was recently used by the Institute of Hydrology at Wallington, England, to apply to a regional station year analysis of floods all over Great Britain. Mr. Alexandria in Australia, who spent his life on flood-frequency distributions, favors the Log Gumbel technique very highly. There are other stations whose curves suggest that this is a good distribution, but I am not going to suggest that the solution is a simple change. Data shown in Figure 2 for 73 years of record upstream on the same river suggest that the Gumbel method is more suitable. Together they illustrate that there is plenty of scope for more research in this area of flood frequency. It is, of course, basic to all drainage design problems.

Secondly either of these figures illustrates another point concerning the risk that dominates our concern of Agnes. On both of the figures I have introduced three scales in addition to the normal return period scale on the bottom. These scales show the percentage chance of exceeding a flood in certain period of years—one for a 50-year period, another 25 years, and the other 10 years. If we look, for instance, to the 100-year value we see that there is a 39 percent chance that that flood will be exceeded in the next 50 years. Other chances are associated with 10 continuous years. Various "return period" floods have different percentages for different design lifetimes.

Another factor that we should give attention to is that some of the theoreticians are pointing out that we should not be working with annual series of floods anymore. We started to work with annual series of floods for ease of punching just one number per year into a desk calculator. But partial-duration series, or methods that include more than one flood each year, hold good promise for improved results. Now that we have computers we can handle a lot of information, and we might be turning our attention toward analyzing more than simply the annual series of floods. There are some data banks at the moment that do give in very convenient form on magnetic tape just annual maximum flood series. We might well be thinking about future research and examine whether we really need partial-duration series or some other basic data and prepare data banks containing that information.

In conclusion, the obvious lesson can be drawn that there seems to be a lot of money put forth for flood relief. We saw the terrible mess that had to be cleaned up after Agnes, which involved a fantastic amount of money, the same amount of money that could build a tunnel under the English channel to France. When it comes to an emergency, we suddenly manage to spend money. Maybe we should be spending a small share of this money on the very matter of research that each panelist has raised. If we invested a very small percentage of these damage funds in research, I do believe our designs would improve and future generations would not have to cough up the huge sums of relief money as at present.

Figure 1. Flood-frequency curve for Susquehanna River.

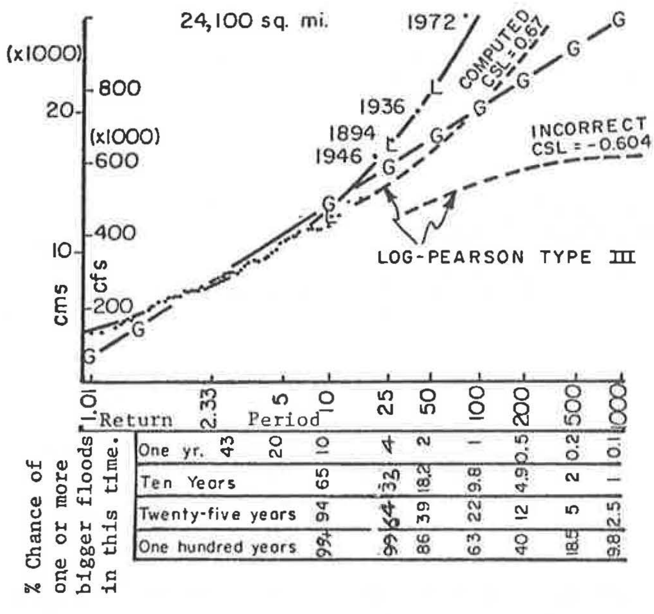
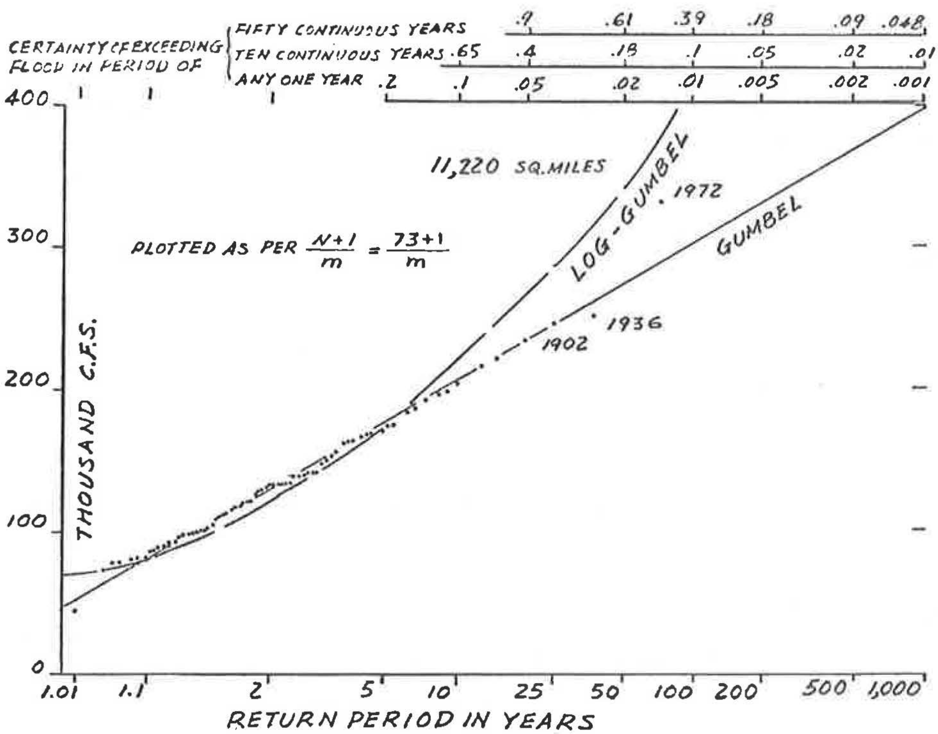


Figure 2. 73-year flood-frequency curve for Susquehanna River.



INFORMAL DISCUSSION

Question

The Weather Service has had a program of broadcasting tornado warnings in the midwest particularly. Lately, they have started announcing flash flood warnings in this area. Is this a program used nationally for, say, hurricanes such as Agnes?

Miller

Yes, the Weather Service has an expanding program in this area. We are developing more expertise all the time. Recently, we have issued flash flood alarms and flash flood warnings. The whole thing is tied up in our ability to more accurately predict precipitation in quantitative amounts. Reaching perfection in this is a long way down the road. We are concentrating in the area where the probability of having flash floods is greatest.

Question

To what extent are automatic flood warning systems being installed—for instance, automatic devices in remote areas that could be placed upstream to give warning that a flood is likely to occur?

Miller

This is part of the flash flood program. There are such automatic devices available. Most of these are put in with the cooperation of the local community. There are a few in, but I do not know exactly where they are located. There must be a cooperative program; they are put in upstream and monitored in police stations and fire houses.

Question

In connection with the rebuilding of these bridges, has there been any change in the hydrologic design, that is, in the amount of water that a bridge can accommodate, or has any change been made in design as a result of the flood?

Bowser

Not directly. Criteria to determine the amount of water reaching a bridge site were changed in 1970.

Question

There are a number of dams on the Susquehanna River. Are these an advantage or a disadvantage to bridges?

Bowser

I am not directly familiar with information to answer that question but, according to the newspaper accounts, lives and property damage were saved because of these dams.

Comment

Where there were completed major dams such as on the Allegheny River in western New York State, which is in the Ohio River watershed, there were tremendous reductions in damage, especially on the Genessee River south of Rochester.

Comment

I would like to mention that most water supply or power dams being kept full cannot do very much to reduce flood flow. In Connecticut, a new dam was to take 3 years to fill. It was filled during Hurricane Diane, and, incidentally, it prevented quite a bit of damage downstream.

Comment

The Kaw Reservoir on the North Carolina-Virginia border reduced the 1972 flood on the Roanoke River in North Carolina; otherwise it would have equaled the 1940 flood, a maximum of record. This dam prevented the flood from getting out of its channel banks.

Question

Were the through lanes on the Interstate System in Pennsylvania closed? If so, for how long?

Bowser

I do not believe that there was any stretch in the Harrisburg area that was closed. However, the state police and National Guard did restrict traffic for 2 or 3 days, principally from going downtown.

Comment

We really need more data in order to determine 50-year floods. Needed, too, are better methods to calculate flood magnitudes and frequencies. Washington should provide the climate to get various local, state, and federal agencies to combine their resources in proportion to their needs so a data base can be provided. We tried this in Montana, but we did not have much success. When the Bureau of Reclamation or the Forest Service or other agencies in the government were asked to work with the Federal Highway Administration and the state highway department to finance the project, we found that it depended on the local administrator's interest rather than on a need to accomplish a common goal. It also depended on timing of each agency's budget, when it is made up and when it is passed by Congress. Therefore, I do not think that it is impossible at the local level to get a program or an agreement in which funds can be combined. I do believe, however, that, if all concerned agencies of the federal government could meet to set the goal and priority, then the local people could get together to determine the best method to accomplish the objectives in their region. It seems that hydrologic regions, and not necessarily state lines, should determine the geographic areas. In this way, we could combine the best efforts in the Corps of Engineers, the Bureau of Reclamation, the U. S. Geological Survey, state highway agencies, and the Federal Highway Administration as well as other interested groups. If we can combine the efforts and talents of these people, then the necessary data base, as well as the best methods for determining magnitude and frequencies of floods, can be developed, and properly designed hydraulic structures will result. What I said is not so much a question as a statement, but in essence it is. If anyone wants to comment on it, I would appreciate being able to report back to Montana that there may or may not be this possibility.

Question

I wanted to raise the same question myself. To rephrase it, what are the chances for interagency cooperation on hydrologic research in order that we can do the job with the least amount of money and combine our efforts to do a better job?

Hofmann

First of all, I think there is quite a bit of this being done although it may not be so apparent or so extensive as it should be. Through the Water Resources Council, or other coordinating groups in Washington, and through direct communication, I think there is a good start for the coordination of activities. Speaking as a federal bureaucrat, the problems associated with the type of coordination you have in mind, of actually integrating the activities at the field level, are tremendous. Each agency has its own mission, and they do not like to see other agencies encroach on that particular mission. And then there are the problems of budgeting. Each agency, because of the unique nature of its mission, has its own priorities, and the individual agency's priorities may differ. The Geological Survey, as its funds are constrained, may decide that the collection of frequency data for small drainage areas is of less priority than the collection of water data on the major river systems of the United States because of the environmental push that the nation is going through. So it is a question of individual agencies selecting priorities, and these may not mesh. So, in answer to your question, I do not see much chance for real improvement in terms of integrated activities at the field level among all of the agencies. I think a more promising action would be to integrate program activities in some fashion and then assign these to specific agencies to carry out in the field, which is more or less what is being done.

Question

With so many agencies collecting water resources data of one type or another, how can the potential user of the data find out what is available?

Hofmann

Well, we are starting a program called the National Water Data Exchange (NAWDEX), by which we are trying to make all water resources data available nationwide from one central location. NAWDEX will not necessarily store all the water information in this one system, but this system will serve as a linking mechanism for data that may be stored in the California Water Resources Department, Water Resources Institutes, EPA, the Corps of Engineers, or the Geological Survey. The concept is that NAWDEX will be the central point where anyone who is interested in water data in a certain locality can go to find out what is available and where to obtain it. NAWDEX is being funded at a very minimal level during this fiscal year. We asked for $2 or $3 million and received $100,000.

Comment

Somebody pointed out that each federal agency has a mission, and perhaps we do not see beyond the end of our nose sometimes. The approach New Jersey used was to call everybody together and say, "We have to live with all you federal agencies, so let's get together and get some kind of common approach and agree on how we're going to operate." I think this is a good approach because the state is going to have to live with the decision so possibly they can act as the coordinating agency to get the federal agencies together and maybe agree on an approach for that state.

Comment

Minnesota has taken that approach. The highway departments and the Federal Highway Administration are putting a lot of money into hydraulic and hydrologic research. Probably whatever we can contribute combined with other agencies as a pool could be used in a little bit better manner and, it is hoped, could provide us with more basic information for the design of facilities.

Comment

Although I agree with the complexity of the problems, I do not think that that is going to be the solution. We have been faced many times in the highway departments and in other agencies where national priorities could not be determined at the local level.

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