TRANSPORTATION RESEARCH
RECORD

No. 1279

*Energy and Environment*

---

# Hydrology and Environmental Design 1990

*A peer-reviewed publication of the Transportation Research Board*

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1990

**Transportation Research Record 1279**
Price: $18.00

Subscriber Category
IB energy and environment

Mode 1 highway transportation

Subject Areas
22 hydrology and hydraulics
23 environmental design
40 maintenance

TRB Publications Staff
*Director of Publications*: Nancy A. Ackerman
*Senior Editor*: Naomi C. Kassabian
*Associate Editor*: Alison G. Tobias
*Assistant Editors*: Luanne Crayton, Kathleen Solomon, Norman Solomon
*Graphics Coordinator*: Diane L. Ross
*Production Coordinator*: Karen S. Waugh
*Office Manager*: Phyllis D. Barber
*Production Assistant*: Betty L. Hawkins

Printed in the United States of America

**Library of Congress Cataloging-in-Publication Data**
National Research Council. Transportation Research Board.

Hydrology and environmental design, 1990.
p. cm.—(Transportation research record, ISSN 0361-1981; 1279)
"A peer-reviewed publication of the Transportation Research Board."
ISBN 0-309-05065-0
1. Roads—Design and construction—Environmental aspects.
2. Flood damage prevention. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1279
[TD195.R63]
388 s—dc20
[625.7'25] 90-26242
CIP

**Sponsorship of Transportation Research Record 1279**

**GROUP 2—DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES**
*Chairman: Raymond A. Forsyth, Sacramento, California*

**General Design Section**
*Chairman: Jarvis D. Michie, Dynatech Engineering Inc.*

Committee on Hydrology, Hydraulics, and Water Quality
*Chairman: J. Sterling Jones, Federal Highway Administration, U.S. Department of Transportation*
*Secretary: Lawrence J. Harrison, Federal Highway Administration, U.S. Department of Transportation*
*Dennis Athayde, Darwin L. Christensen, Stanley R. Davis, David J. Flavell, Thomas L. Hart, John Owen Hurd, S. Bennett P. John, Kenneth D. Kerri, M. Dean Knighton, Floyd J. Laumann, Norman Miller, Glenn A. Pickering, Don L. Potter, Brian M. Reich, Jean Reichert, Everett V. Richardson, Verne R. Schneider, Robert F. Shattuck, H. Earl Shaver, Michael D. Smith, Corwin L. Tracy, E. L. Walker, Jr., Ken Young, Michael E. Zeller*

Committee on Landscape and Environmental Design
*Chairman: David H. Fasser, New York State Department of Transportation*
*Secretary: Barbara M. Schaedler, State University of New York at Farmingdale*
*Charles R. Anderson, Jay W. Bastian, E. Leroy Brady, Nicholas R. Close, Harold D. Dolling, Lawrence E. Foote, Robert L. Jacobsen, Eugene Johnson, Edward N. Kress, Harlow C. Landphair, Charles R. Leer, Win M. Lindeman, Hans A. Littooy, Derek Lovejoy, Charles P. Monahan, Hubertus J. Mittman, Carroll L. Morgenson, Barbara Petrarca, Elga Ronis, Jacquelyn A. Ross, Richard C. Smardon, Howard R. Wagner, Clyde D. Walton*

**GROUP 3—OPERATION, SAFETY, AND MAINTENANCE OF TRANSPORTATION FACILITIES**
*Chairman: H. Douglas Robertson, University of North Carolina–Charlotte*

**Maintenance Section**

Committee on Roadside Maintenance
*Chairman: Wayne G. McCully, Texas Transportation Institute*
*Dorothy L. Andres, Suzanne W. T. Batra, Robert L. Berger, John P. Burkhardt, Lonnie M. Cargill, William E. Chappell, Joseph M. Dipaola, Edward C. Dixon, Turney J. Hernandez, Roy R. Johnson, S. Joseph Lesley, J. A. Lewis, Percy B. Middlebrooks, Jr., Richard C. Moffett, D. James Morre, George P. Romack, Ronald G. Stahl, Robert E. Tatman, Clifton R. Taylor, Carl B. Wells, Bernard G. Williams, Ross M. Wolfe*

George W. Ring III and Frank N. Lisle, Transportation Research Board staff

Sponsorship is indicated by a footnote at the end of each paper. The organizational units, officers, and members are as of December 31, 1989.

# Transportation Research Record 1279

## Contents

# Foreword

This Record includes papers on hydrology and hydraulics, landscape and environmental design, and roadside maintenance.

Wacker reports on the lessons learned and the recommendations that emerged following the August 1985 flash flood that devasted Cheyenne, Wyoming. Naghavi and Allain discuss several attempts by the Louisiana Department of Transportation and Development to protect the shoreline and Louisiana Highway 82. Successes and failures of shore protection efforts were evaluated to identify the important factors needed for the proper design of shore protection structures. Thompson summarizes Federal Highway Administration (FHWA) assistance in the National Transportation Safety Board analysis of the collapse of the northbound US-51 bridge over the Hatchie River in Tennessee. The bridge site, field observations, stream stability, analysis of aerial photographs, model studies, and foundation analysis are discussed. Seybert and Kibler present the capabilities of the Pennsylvania State Urban Hydrology Model. The model contains 17 subprograms that perform various hydrologic and hydraulic operations, which include curve number weighting, travel time, swale design, hydrograph combining, graphics plotting, rainfall calculations, Soil Conservation Service (SCS) unit hydrograph, SCS tabular hydrograph, modified rational hydrograph, Muskingum channel routing, modified Puls routing, and multiple-state outlet analysis.

Chong and Lumis report on experiments to evaluate several spray treatments and their effect on reducing salt damage in peach trees. Evink reports on research to develop measures that provide safe crossings of I-75 (Alligator Alley) for the endangered Florida panther. Although crossings and fencing have not been finished, panthers and other animals are currently passing through these structures, indicating that these crossings are being used. Johnson discusses the impacts of human-caused disturbance on the nesting success of bald eagles in Southeast Alaska. It includes a review of the literature on disturbance of raptors generally and bald eagles specifically.

Smith and Lord summarize the more than 15 years of water quality research that the FHWA and state highway agencies have supported. They discuss the need for further research in this area and possibilities for accomplishing this research.

Gouveia describes a maintenance-based design approach for roadsides on the urban expressway system in Illinois. The goals of the planned-management roadside design include reduced construction costs, reduced maintenance efforts, and improved public acceptance. Bolin et al. discuss the decline of native prairie vegetation in Minnesota reported in recent surveys and an innovative and cooperative roadside prairie preservation and management program. Lyman and Gover investigated the performance of 50 wildflower species for use on Pennsylvania's roadsides. Each species was evaluated for percentage cover, percentage weed invasion, and percentage of the plot covered by blooms. The wildflowers were planted in Spring 1988 and their performance is evaluated through 1989. Narasimhan et al. describe a computerized system for the storage, organization, analysis, and display of field collected scour data. The system accepts input from the user, and on the basis of user specification, allows users to organize and view data in many formats. Naghavi et al. compare three moment-based estimation procedures (direct moments, log-transferred moments, and mixed moments) using observed stream data samples from Louisiana and its neighboring states. No method performed in a clearly superior manner across the entire range of data.

# Flash Flood: Highway-Related Lessons and Recommendations

A. MAINARD WACKER

On August 1, 1985, a flash flood devastated Cheyenne, Wyoming. Although the flood receded to less than bankfull in less than 3 hr, 12 people lost their lives and damages exceeded $65 million. Emergency operations were severely curtailed; officials were unable to immediately react; and essential services provided by hospitals, police, and fire departments were forced into abeyance. Significant flood hazard areas emerged where none were expected. From this disaster, many lessons were learned and recommendations emerged regarding flash floods and their related hazards.

Cheyenne, a community in the southeast corner of Wyoming, has a population of about 50,000. It is located on the western high plains at an elevation of about 6,100 ft with the surrounding terrain consisting principally of rolling prairies. The Laramie Range foothills are about 30 mi to the west.

As shown in Figure 1, three major drainages enter Cheyenne: Clear Creek, Crow Creek, and Dry Creek. Clear Creek has its confluence with Crow Creek in southwest Cheyenne. Because flows are interdicted by a large stock and irrigation pond located immediately west of Interstate 25, Clear Creek is essentially an intermittent stream within the Cheyenne city limits. Most flows below the stock pond are caused by localized storms, with some small return flows occurring from residential watering of lawns. It is not believed that significant flows have, as yet, bypassed the large stock pond and caused major flooding. Clear Creek did not cause significant flooding during the Cheyenne flood.

Crow Creek passes through Cheyenne from west to east. It is a perennial stream having its headwaters in the Laramie Range. Three large water supply and recreational reservoirs restrict the passage of flood waters originating in the upper watershed. Crow Creek has a history of conveying major floods into Cheyenne, as presented in Table 1. These floods result primarily from storms located between Cheyenne and the reservoirs. However, on occasion snowmelt has produced large floods and has been the primary source of the annual runoff. Several Cheyenne storm drains outfall into Crow Creek. Thus, storms in the Cheyenne urban area can also cause flooding along Crow Creek. Accordingly, before the Cheyenne flash flood, it was recognized that large floods in Crow Creek had occurred before and could be reasonably expected again.

Dry Creek enters the northwest corner of Cheyenne and flows southeasterly, ultimately having its confluence with Crow Creek east of Cheyenne. Except for some return flows generated by residential lawn watering, Dry Creek is essentially just that—dry. Within the city limits, there are several minor retention ponds, as well as two major detention ponds. The pond locations are shown in Figure 1. These major detention ponds were constructed in the early 1980s. One of the major detention ponds is located near the headwaters of the North Fork of Dry Creek on Francis E. Warren Air Force Base (WAFB) lands. This pond was constructed by a developer to help mitigate potential runoff increases from adjacent land development. The other pond, Carey detention pond, was constructed by the city to mitigate potential flood hazards discovered during the design of a cooperative city and Wyoming Highway Department (WHD) street project. Before August 1, 1985, Dry Creek flooded only in one reach, on August 2, 1966. This historical flood was caused by a localized storm (Table 1).

Wyoming Highway Department, P.O. Box 1708, Cheyenne, Wyo. 82002-9019.

## PREFLOOD ACTIONS

Before the Cheyenne flood of 1985, the city was aware of serious Dry Creek flood hazards requiring attention. The WAFB pond and Carey detention pond reflect this awareness, as did flood plain clearing on Dry Creek immediately upstream from U.S. Highway 30 in east Cheyenne. Discussions were under way between the city and the WHD regarding the construction of additional Dry Creek ponds immediately west of Cheyenne on WAFB lands.

A minor city and WHD cooperative traffic-related project in the 8th Avenue, Central Avenue, and Warren Avenue area was completed before the Cheyenne flood. During the design of that project, it was discovered that there might be a significant flood hazard just beyond the southern project limits in a low, poorly drained area located within a residential neighborhood near the intersection of Pioneer and 7th Avenues. Notably, this depression was not associated with a drainageway or flood plain.

Elsewhere in Cheyenne, the WHD had identified a flood hazard area in southeast Cheyenne. The hazard had been mitigated by the construction of the Sun Valley detention pond and the Lincoln Highway detention pond.

Because flood warning signs were flashing, the city with its limited resources tried to respond. Ordinances were passed to prevent developers from exacerbating the existing flood hazard problems. A task force of flood professionals and other experts had been appointed by the mayor to provide recommendations. This task force met for several years and culminated its findings by recommending that a city master drainage plan (MDP) be developed. At the time of the 1985 flood, the city was in the preliminary stages of selecting a contractor to develop an MDP. Unfortunately the city was too late, and disaster overtook it.

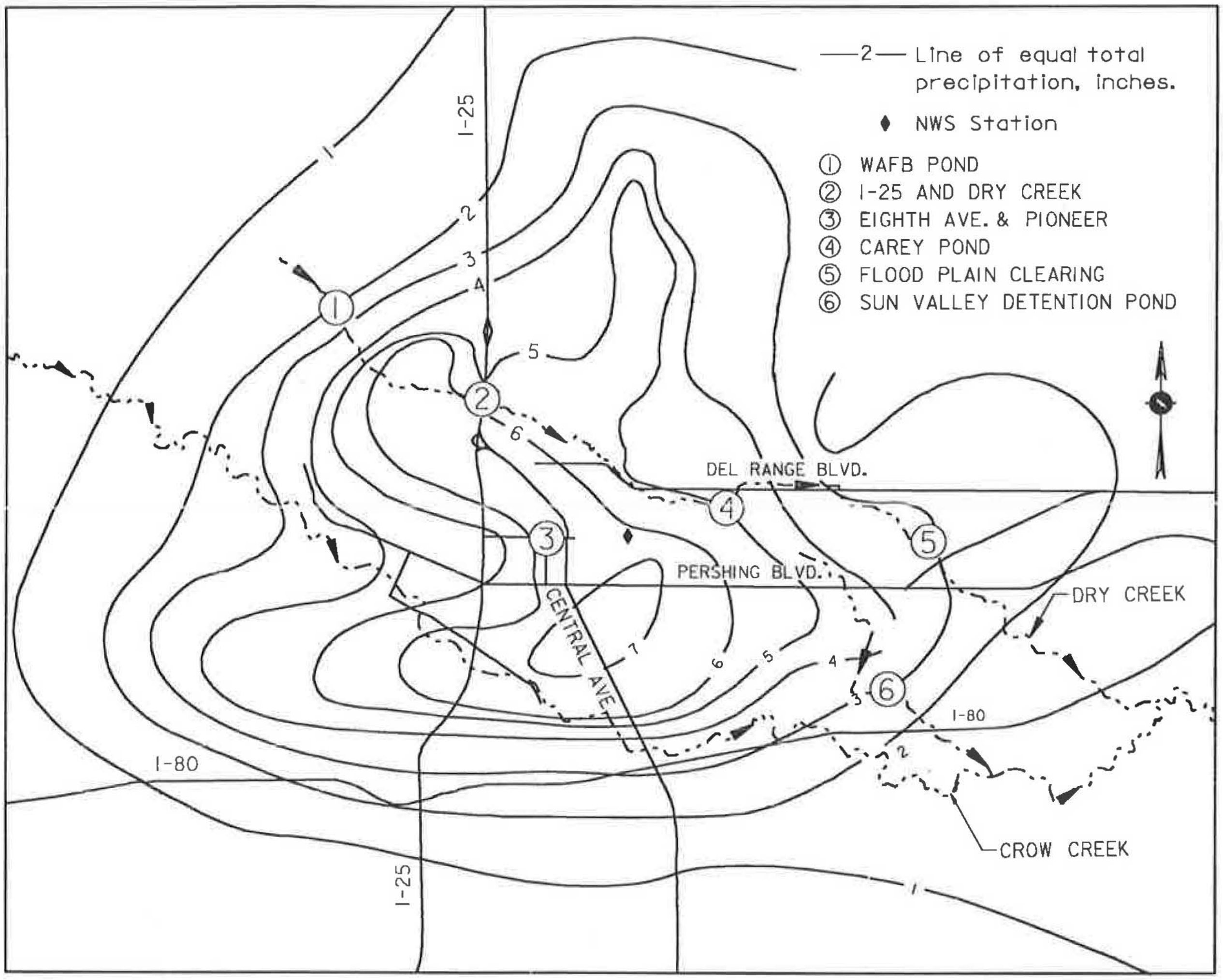



**FIGURE 1 Topography of subject area.**

## METEOROLOGY AND HYDROLOGY

The eastern front range of the Rocky Mountains has been identified as a flood hazard area (*1*) because of the numerous intense storm cells traveling along the front range each year. In Colorado, several disastrous floods have occurred in the past 20 years: one in the vicinity of Denver, and the other within the canyon reaches of the Big Thompson River west of Loveland (*1*). During this same time, there was a disastrous flood in Rapid City, South Dakota. This flood was also the result of an intense storm cell west of that community.

## FLOOD HISTORY

On the basis of the history presented in Table 1, Cheyenne was aware of an adverse flood history along Crow Creek. Conversely, the city and the WHD had limited information regarding floods along Dry Creek, primarily only a 1965 flood in an upper reach of Dry Creek which caused no significant damage (see Table 1). There was no known flood history for Clear Creek.

The August 1, 1985, storm resulted in large flood discharges on Crow Creek, as indicated in Table 2. Although these discharges nearly tripled in magnitude from the west to the east side of the city, flood damage was relatively minor.

The Cheyenne flood did jar the memories of several local residents. From their recall and subsequent study (*2*) by the U.S. Geological Survey (USGS), a more complete flood history for Dry Creek surfaced. The flood magnitudes for Dry Creek are presented in Table 2. On Dry Creek, 11 of the 12 lives lost occurred because of flooding along Dry Creek. Significant flood damage occurred on all reaches of Dry Creek as well.

From Tables 1 and 2, comparison of the flood discharges at the I-25 crossing of Dry Creek indicates that the Cheyenne flood of August 1, 1985, was about 20 times greater than the measured discharge of 300 $ft^3$/sec in 1965. Could this amount of increase have been reasonably expected? Investigations by the WHD before the Cheyenne flood suggest that an increase of six to eight times the 1965 flood might occur with 100-year frequency. A 20-fold increase would probably have been considered a possible, but not a practicable, design criterion.

Severe flooding in the Cheyenne area commonly occurs from intense, short-duration storm cells. Precipitation occurs

TABLE 1 MAXIMUM DISCHARGES AT SELECTED SITES NEAR CHEYENNE

| Stream and location | Date | Maximum discharge (cubic feet per second) | Drainage area (square miles) | Source |
|---|---|---|---|---|
| Crow Creek (at Cheyenne) | May 20, 1904 | [1]8,500 | -- | 1 |
| Crow Creek (sec. 36, T. 14 N. R. 67 W., 2 miles west of Cheyenne) | June 2, 1929 | 8,200 | -- | 1 |
| Crow Creek (sec. 19, T. 14 N. R. 68 W., about 12 miles west of Cheyenne) | June 2, 1929 | 3,600 | 102 | 2 |
| | June 5, 1929 | 2,000 | | 2 |
| Crow Creek (at 9th Street, Cheyenne) | July 22, 1983 | [1]2,500 | -- | 2 |
| Dry Creek (upstream from Interstate Highway 25, Cheyenne) | August 2, 1966 | 320 | -- | 3 |
| Dry Creek (at Seminoe Road, Cheyenne) | August 2, 1966 | 213 | 3.0 | 4 |
| Lodgepole Creek (sec. 26, T. 16 N. R. 67 W., 13 miles north of Cheyenne) | June 2, 1929 | 1,000 | 122 | 2 |
| North Fork Muddy Creek (25 miles east of Cheyenne) | September 1, 1966 | 5,460 | 15 | 4 |
| Porter Draw (at U.S. Highway 85, 5 miles south of Cheyenne) | May 1, 1977 | 944 | 7.60 | 5 |
| Simpson Creek (at Highway 85, 8.5 miles south of Cheyenne) | July 24, 1977 | 1,080 | 3.29 | 5 |

[1] Estimate

[Source: 1, U.S. Geological Survey, 1964; 2, U.S. Geological Survey, unpublished records; 3, A.M. Wacker, Wyoming Highway Department, oral commun., 1985; 4, U. S. Geological Survey, 1967; 5, U.S. Geological Survey, 1978]

TABLE 2 MAXIMUM WATER SURFACE ELEVATION AND DISCHARGE DATA FOR THE FLOOD OF AUGUST 1, 1985

| Site number and location | Drainage [1]area (square miles) | Average maximum water-surface elevation (ft) above sea level) | | Maximum Discharge (cubic feet per second) | Unit discharge (cubic feet per second per square mile) | Approximate time of maximum discharge (Mountain Daylight Time) |
|---|---|---|---|---|---|---|
| | | Upstream from (at) location | Downstream from location | | | |
| **Crow Creek** | | | | | | |
| 1-upstream from Interstate Highway 25 | 252.8 | 6,074.2 | -- | 2,300 | (2) | 8:00 - 8:30 p.m. |
| 2-at 19th Street | 257.1 | 6,054.7 | 6,053.2 | 2,980 | (2) | 8:45 - 9:15 p.m. |
| 3-at Warren Avenue | 294.1 | 6,023.9 | -- | 4,460 | (2) | 9:30 - 10.00 p.m. |
| 4-at Morrie Avenue | 295.6 | 6,011.9 | 6,008.9 | [3]8,260 | (2) | 10:15 - 10:45 p.m. |
| 5-at Interstate Highway 80 | 296.4 | 5,980.9 | -- | 7,470 | (2) | 10:30 - 11:00 p.m. |
| **Unnamed tributary at Crow Creek** | | | | | | |
| 6-at College Drive | 4.35 | -- | -- | 3,540 | 814 | 9:00 - 9:30 p.m. |
| **Dry Creek** | | | | | | |
| 7-at Buffalo Avenue | .95 | -- | -- | 3,820 | 4,020 | 7:30 - 8:00 p.m. |
| 8-at Interstate Highway 25 | 1.88 | 6,146.5 | 6,137.5 | [4]4,960 | 2,640 | 8:00 - 8:30 p.m. |
| 9-at Yellowstone Road | 2.64 | 6,125.6 | 6,117.6 | [5]5,820 | 2,200 | 8:00 - 8:30 p.m. |
| 10-at Powderhouse Road and Dell Range Boulevard | 3.76 | 6,086.8 | 6,075.6 | [6]4,080 | 1,090 | 8:30 - 9:30 p.m. |
| 11-upstream from College Drive | 6.67 | 5,996.1 | -- | 5,880 | 882 | 9:00 - 9:30 p.m. |
| 12-at East Pershing Boulevard | 9.27 | 5,955.6 | 5,953.7 | [7]4,310 | 465 | 11:15 - 11:45 p.m. |

[1] Part of which may be non-contributing.
[2] Not computed, most of drainage area did not contribute to storm runoff.
[3] Includes 4.240 cubic feet per second through culverts-assumed to be clear of debris.
[4] Includes 640 cubic feet per second through culverts-assumed to be clear of debris.
[5] Includes 470 cubic feet per second through culverts-assumed to be clear of debris.
[6] Includes 260 cubic feet per second flowing east along Dell Range Boulevard.
[7] Includes 580 cubic feet per second through culverts-assumed to be clear of debris.

either as rain or as rain and hail. Flooding from low-intensity, widespread storms is relatively rare—unless perhaps the storms occur on a large snowpack. Crow Creek has produced long-duration flood stages at 100-year frequency from snowmelt occurring in the Laramie Range.

## STORM CELLS

On August 1, 1985, the National Weather Service (NWS) was tracking with its radar facilities several potentially dangerous storm cells located along the front range of the Rocky Mountains, as well as further east and south on the high plains. As reported by Chappell and Rodgers (*3*), one of these storm cells

> began as an east-west multicellular system just south of the city near the summit of (what is termed the) Cheyenne Ridge. This system developed in a conditionally unstable air mass that formed over southeast Wyoming, as a southeasterly flow of very moist air at low levels became juxtaposed with an area of steepening lapse rates to the west. Early cells drifted slowly northward in agreement with the pressure-weighted vector mean wind of the environment. New convective growth on the southwest flank of this multicellular system eventually produced a wave-shaped convective system, which rapidly developed supercell structure. As the supercell began to rotate, the storm became stationary over the city for nearly 2 hours. This lack of motion is believed to have been due to helicity, which promoted the transverse propagation of the supercell's updraft at a rate that counteracted the effects of the vector mean wind of the environment. The storm began to move southeastward with the arrival of a short-wave trough and soon dissipated as it encountered increasingly stable conditions. The results of the study suggest that the eastern foothills of the Rocky Mountains and the adjacent high plains may be particularly vulnerable to this type of storm. Deep convection frequently occurs over this area when moist air arrives from the Great Plains, driven by a low-level easterly jet. The combination of strong low-level easterly flow topped by weak middle-level southerly flow can apparently produce sufficient wind shear for supercell formation, while producing a vector mean wind for the environment that gives little or no eastward motion relative to the ground.

The storm cell top reached above 60,000 ft. Rain and hail began falling about 6:20 p.m. and lasted until 9:45 p.m., growing in intensity during that period. The maximum 1-hr precipitation was 3.51 in., occurring between 8:00 and 9:00 p.m., as shown in Figure 2. Winds up to 70 mph accompanied the storm. The total precipitation set a new 24-hr record. Hail up to about 2.5 in. in diameter fell in portions of the city, reaching depths of about 1 ft. Runoff carried hail into low-lying areas where it collected into drifts 3 to 6 ft high. Funnel clouds were also observed during the storm, and two short-lived tornados were reported.

The hyetograph of the storm is considered unusual, as compared to the hyetographs commonly used for engineering design purposes. Studies have also suggested that the hyetograph was not a common one (*4*). Most intense storm cells on the western high plains commonly generate hyetographs with relatively advanced peaks (*4*). The August 1 hyetograph had a delayed peak as shown in Figure 3.

A rainfall distribution pattern for the storm cell was developed from a bucket survey by the USGS (*2*). This pattern is shown in Figure 1. Essentially, the storm was centered over the Wyoming State capitol building, where about 7 in. of rain

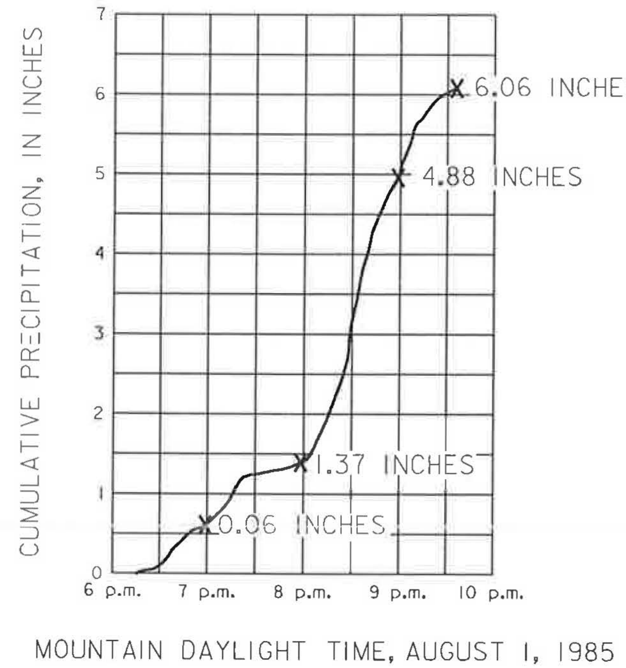



**FIGURE 2 Precipitation at station near storm center.**

is estimated to have fallen in 2 of the 3 hr. As indicated by Figure 3, over half of this 7 in. fell in about 1 hr. The officially recorded total rainfall was 6.06 in. at the NWS station at Cheyenne, which was about 1 mi from the storm's center.

The maximum probable rainfall for an intense storm cell in the Cheyenne area is believed to be about 14 in. Studies by the USGS (*5*) were made using the precipitation data from the August 1 storm. The findings show that the occurrence of this storm over Cheyenne was less likely than that of a 100-year event—perhaps even than that of a 500-year event. The frequency of the Cheyenne flood is uncertain, as storm frequency and flood frequency are not necessarily the same.

A tornado was sighted southeast of Cheyenne immediately before the storm. This funnel was spawned by the same meteorological conditions that generated the killer storm cell. This unfortunate coincidence resulted in an unnecessary death.

## POST-FLOOD ACTIONS

The probable frequency of the storm and resultant flood has become a significant issue in the numerous lawsuits following the flood. Recognizing this issue, the WHD commissioned the USGS to undertake a thorough and complete study of the flood. This request was timely, as it was made even as the storm raged and the flooding was occurring. The WHD also expected other issues to be raised in the inevitable lawsuits. Because of them, the WHD requested the USGS to

- Identify the travel times for the flood peaks as they passed downstream (for later use in calibrating storm and flood runoff models),
- Identify flood depths and discharges throughout Cheyenne to include runoff sources and direction,
- Identify any previously unknown flood history, and
- Document findings in a defensible report.

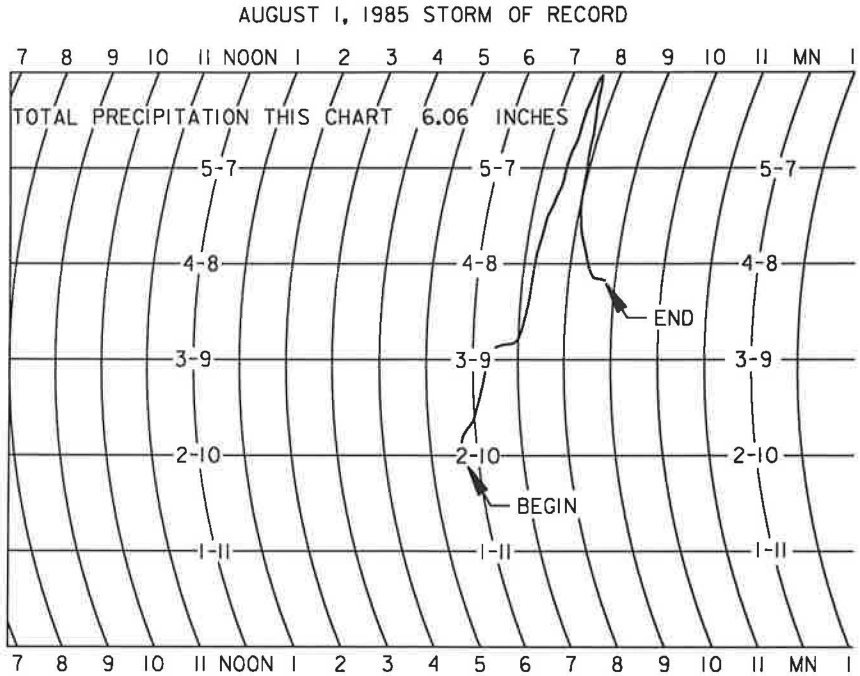



**FIGURE 3 Hyetograph of storm with delayed peak.**

In retrospect, the WHD should have requested the USGS to identify the probable recurrence range of the storm as well as of the flood. However, subsequent studies by the USGS did fulfill this need (*5*).

In order to facilitate the USGS study, the WHD used black-and-white as well as color infrared aerial photography. These photographs were obtained before noon on the day following the flood to optimize the recovery of highwater marks. In order to ensure that the USGS study was well focused on expected legal problems, the WHD had teams of experienced hydraulic engineers on the ground by 8:00 a.m. the day following the flood. These teams took ground photographs, documented their observations, and identified potential legal problems. In consultation with the WHD, the USGS study documented peak discharges expected to be hydrologically pertinent in legally sensitive reaches wherever possible. In some instances, the local terrain precluded an indirect measurement. In addition, highway elevations and the approximate time the peak arrived in the reach were obtained. Table 2 indicates these flood-related findings. These efforts resulted in the publication of a creditable and useful report (*2*).

## FLOOD WARNING LESSON

As a result of a disastrous tornado in 1979, Cheyenne had installed an early-warning system for tornados before the flood. The NWS would provide the city's emergency command center (ECC) with tornado watch and tornado warning conditions. Under watch conditions, the police are assigned to strategically located observation posts where they watch for descending funnel clouds. A siren system throughout Cheyenne provides a tornado early warning, along with broadcasts from television and radio stations.

On August 1, 1985, both before and during the storm, the sirens warning people of a tornado in the Cheyenne area were sounded. However, because of the intense downpour and attendant heavy electrical storm activity, the sirens did not function properly and sounded only sporadically. Those people neither listening to their radios nor watching their television (most knowledgeable people disconnect their television during electrical storms) thought the sirens were merely malfunctioning and, fortunately, did not go to their basements. Unfortunately, one elderly lady went to her basement seeking cover from the tornado. She lived near the intersection of Pioneer and 7th Avenues, which, as noted earlier, was only a suspected flood hazard area—just a shallow topographical depression within an affluent, older residential neighborhood. As this depression quickly filled, flood waters came crashing through the basement windows and filled the basement. The basement filled too fast for her to open the door to the upstairs—unfortunately the door opened inward. She drowned. One can only wonder what the death toll might have been had the tornado actually destroyed buildings in which people had sought shelter.

The city, in cooperation with the USGS and NWS, has now installed a flood early-warning system. The system senses precipitation and flood levels at key locations, and automatically transmits this information to the NWS for interpretation and action. What actions have been taken for warnings to discriminate between the natures of potentially imminent disasters is unknown.

## FLOOD HAZARD TOPOGRAPHY LESSON

Federal Emergency Management Agency (FEMA) studies had identified potential flood hazard areas along Crow Creek and Dry Creek. The Cheyenne flood verified these findings. However, most of the flood damage and two deaths occurred in unsuspected flood hazard areas.

First, it is necessary to consider the evolution of storm drain design in most high plains communities to understand the topography problem. Acceptable design philosophy in the first half of this century required that some nominal storm drain system be provided for frequent, low-intensity storms, and for snowmelt. Runoff from large storms was expected to temporarily block streets as it sought a suitable outfall channel. This design philosophy is still acceptable. However, actually determining what occurs during major runoff events was commonly not part of this earlier design philosophy—unless perhaps there had been a history of major flood damage or loss of life. Further, land development was not strictly regulated. Commonly, this was the fault of the political system as communities sought to encourage development for its contingent jobs and expanded tax base. In defense of our engineering predecessors, sufficient technology, including long-term and detailed meteorological data and flood histories, to adequately design storm drain systems was not available. Even had the meteorological data been available, the technology to translate this data into runoff rapidly and reliably was, as yet, either unknown or not in common use. Sophisticated rainfall runoff computer models and aerial photography and mapping were not even envisioned. Because of this lack, and perhaps for political reasons as well, detailed drainage designs were not made by engineers—or required by communities. Significant topographical flood hazard areas often went undetected because they did not involve a major or recognizable waterway or flood plain.

Cheyenne had several of these insidious, heretofore undetected, topographical flood hazard areas. One such area was in the residential neighborhood mentioned earlier, where the elderly lady was drowned in her basement. This shallow (2- to 3-ft) depression encompasses portions of several blocks and, on the ground, looks like a typical upper-middle-class neighborhood where flood waters can pass through as surface flow and not collect to cause a significant flood hazard; that is, the depression is not readily apparent. A small drainage system collects and removes low flows and temporarily ponds higher flows on lawns. From all appearances, storm water would flow out of the area before a serious flood hazard occurred. Fifty years ago, had current technology been available along with a design philosophy that requires an evaluation of the hazards from large flows and had there been a responsive political system, this hazard could have been avoided—and at much less cost. Before August 1, 1985, only some citizen complaints and drainage-related findings associated with the traffic project at 8th Avenue provided any early warning signs. However, the Cheyenne flood demonstrated that drainage collecting in an unobstrusive depression can cause severe flood damage and loss of life. Litigation studies by the WHD and others following the Cheyenne flood have shown that the storm was so intense that the resultant hazard would have occurred regardless of the contributing drainage area—just from precipitation falling within the limits of the depression alone. In cooperation with the WHD, the city has completed a storm drain project costing over $3 million to accommodate a 100-year flood occurring in this low area. The 100-year flood is smaller than the flood occurring on August 1, 1985. This flood hazard could have been avoided 50 years ago at little or no cost had better engineering been possible and a more stringent regulation enforced. The city is preparing to explain this in court at this writing.

Another unrecognized terrain hazard was downtown Cheyenne. Storm water runoff from the northwesterly portions of Cheyenne lying in the Crow Creek basin moves through storm drains and into downtown Cheyenne as street flow in attempting to reach Crow Creek (see Figure 1). Street flooding and relatively minor and localized basement flooding in downtown Cheyenne had occurred in the past. The degree to which the Union Pacific Railroad (UPRR) yards interdicted this street flow, forcing it into the existing storm drain system, apparently was not recognized; nor was it recognized with the large amounts of runoff that occurred with the Cheyenne flood that the storm drain may not be adequate and that flood waters would have to pond until they could escape by some alternate route. In the Cheyenne flood, this ponding had to reach the tops of the parking meters to find an adequate alternate route out of the downtown area. This water level resulted in significant flood damage to commercial property throughout downtown Cheyenne. Foundation problems suspected as being related to the flood are just now beginning to emerge. Additionally, in seeking alternate escape routes flood waters were diverted into heretofore unaffected areas. In retrospect, only with a careful study of the topography afforded by modern aerial methods; the use of a sophisticated storm drain computer model; and the availability of meteorological data for a stalled, intense, high-volume storm would it have been possible to forecast these topographical ramifications for an event such as the August 1 storm.

## OTHER FLOOD HAZARD LESSONS

### Curb Overtopping

In smaller, less-populated areas, architects, engineers, and those responsible for issuing and regulating building permits are often not familiar with storm drainage technology and fail to fully recognize the flood hazards associated with curb overtopping. The below-grade location of off-street parking, critical records storage, hospital services, underground parking, costly mechanical equipment, and, most important, the ECC in Cheyenne proved to be disastrous. Once curb overtopping occurred from street flows, these facilities were flooded. Because this flooding occurred during nonworking hours, there was no loss of life, but the majority of Cheyenne's flood damage resulted from, and the ECC was lost because of, this effect.

### Curtailed Access

The Cheyenne flood isolated segments of the community by interdicting main transportation corridors. City officials were delayed or prevented from directing emergency action—the loss of the ECC exacerbated this situation. Human life was placed in jeopardy when portions of Cheyenne were isolated by flood waters. A man was suffering a heart attack brought on by bailing flood waters out of his basement window well to prevent flooding of his basement. Emergency vehicles could not reach him. He died. The city has now identified emergency transportation corridors and elected to ensure that vehicle passage will be possible up to a 500-year flood event. The risks associated with even larger floods were subjectively determined to be acceptable. Perhaps some type of economic risk analysis would have aided in this decision. In any event, future plans are to upgrade these drainage structures.

### Overtopping

Many people in urban areas do not recognize the hazard associated with flood waters that overtop a roadway or street. Young people joyriding through overtopping flood waters and parents anxious to reach home to check on their children during a disaster may be oblivious to such flood hazards. People leaving theaters and not realizing that a severe storm and dangerous flooding were occurring were placed in jeopardy. Because of these particular circumstances, apparently (we'll never really know) nine people drove into what only appeared to be street flooding, but in fact was overtopping flows. They all drowned. Because of overtopping, friction is lost between vehicle tires and the road. Most cars become buoyant once flood waters reach the underbody (not very high for many cars). Because of the momentum from overtopping flood waters, vehicles are easily swept off the road into deep, fast-moving waters, or into drainage structures, where they become lodged. Heroic efforts may then be needed to rescue people from errors brought on by ignorance, stupidity, or poor judgment during efforts to escape. A sheriff's officer who attempted to rescue some of these nine people also drowned after his safety rope broke.

Not all overtopping is to be avoided. Sometimes overtopping is essential for protecting property or securing cost-effective structures. However, where overtopping is expected, measures to ensure controlled road closures are essential—particularly in urban areas.

### Guardrails

Flood waters backing up behind a road or highway may cause a hazard either through flooding of property, or by diversion of flows. As bad as this was at some sites, it could have been worse in the Cheyenne flood. Flood waters overtopped Interstate 25 where it crossed Dry Creek and backed into an affluent, residential area. There was an Interstate safety project under way at this location. In 2 weeks, solid concrete safety barrier was to be permanently placed in the median, across where the flood waters had overflowed the highway. Had the flood occurred 2 weeks later, the upstream flood waters would have been 4 ft deeper, caused much greater property damage, and in all probability, loss of life. Open barriers, such as box-beam or ARMCO types, have also caused similar problems, particularly where drift and debris are present. When the need for a safety barrier at this location was subsequently reassessed, it was determined that it was not needed. If a barrier is essential to public safety, lower gradelines or safety fill slopes should be considered.

### Fencing

Chain link right-of-way fence did cause a temporary backwater condition at this same Interstate 25 crossing of Dry Creek. This condition was caused by debris collecting on the fence. The fence post eventually failed, thereby allowing flows to pass more freely.

### Documentation

Some engineers prefer to avoid design documentation because they feel it is incriminating. Experience following the Cheyenne flood suggests otherwise. To date, the WHD, the city, Laramie County (the county), and in one case, the UPRR have been individually or collectively involved in five lawsuits resulting from the Cheyenne flood. In all of these suits, the WHD provided expert testimony. The WHD was named in three of these suits, successfully defended itself in two of them (although one has been appealed by the plaintiff to the state supreme court), and expects to be dropped from the third. Complete documentation proved state-of-the-art practices were used at the time of the design, and that the findings were reasonable. This either aided the WHD in its defense or convinced the plaintiff that little would be gained in naming the WHD. Thus, one of the lessons learned is that the value of thorough and complete design documentation should not be underestimated.

### Practices

Documented WHD designs for the litigated sites, while not reflecting state-of-the-art practices by current standards, were state of the art at the time of design. The use of current standard practices proved effective in refuting a common plaintiff claim of negligence. In retrospect, it would have been even more helpful if some practice-related design decisions and criteria determinations reached at joint meetings between the WHD and the city had been documented. Personal recall after many years is suspect, and subject to attack in court.

### Professionalism

In two suits where the WHD was not named, failure to involve experienced hydraulic engineers in a drainage design apparently led to judgmental errors. One defendant lost a suit, in part, because an experienced and licensed professional hydraulic engineer was not involved in the design.

### Policy

Some agencies involved in public works prefer to avoid promulgating a complete, definitive drainage policy because they feel it can be incriminating—or perhaps a waste of time. Experience following the Cheyenne flood again suggests otherwise. One of the defendants lost general credibility in court because of lack of a definitive drainage policy for establishing design criteria. Policy should also address internal operating procedures and decision-making responsibility. This same defendant lost credibility because of these particular omissions as well. Policy should also reflect the use of state-of-the-art practices at the time of design, either by inclusion or reference. The need for documentation should also be a policy issue.

## CORRECTING FLOOD HAZARD DEFICIENCIES

Highway agencies or communities that have allowed flood hazards to evolve usually do not find simple, inexpensive solutions. However, to ignore such problems only ensures the time when lives will be lost and costly damages will occur. At

TABLE 3 LESSONS AND RECOMMENDATIONS

| No | Lesson | Recommendations |
|---|---|---|
| 1 | Little or no action by either a Highway Agency, City or County to estimate flood hazards ultimately cost lives, results in excessive property damage, and is probably not defensible in court. | Cities and Counties must<br>° jointly develop, maintain, update and implement a master drainage plan,<br>° devise a funding mechanism to implement the plan, and<br>° recognize that flood hazards transcend political boundaries. |
| 2 | Lack of a single, state of the art ordinance to regulate development in flood hazard areas (both along waterways as well as from street or sheet flow), and to fully enforce such ordinances is not defensible in court. | Cities and Counties must<br>° jointly develop and enforce a state of the art ordinance to regulate development, and<br>° include <u>all</u> flood hazard areas (not just channels). |
| 3 | Lack of a comprehensive and effective early warning system is not defensible in court. | Cities must<br>° develop a reliable (under all natural and man-made disaster conditions) early warning system,<br>° insure the system clearly discriminates between the type of disaster, and<br>° provide ongoing public education as well as routine testing of the system. |
| 4 | The best emergency management plan cannot be implemented if decision making officials or their designates cannot reach the ECC. | Cities and counties must<br>° identify key routes necessary to avoid compartmentalization of a community during a flood,<br>° identify flood hazard areas along these routes, and<br>° provide drainage facilities to insure the movement of vehicles during a flood -- probably at the 500-year or possibly even the Probable Maximum Flood level. |
| 5 | An ECC cannot function well when under water. | Locate ECC's where they<br>° are secure from natural and man-made disasters,<br>° can be quickly and easily reached (see 4 above), and<br>° are self contained so as to accommodate such things as power outages. |
| 6 | Disaster planning and practice could have saved lives. | Cities and Counties should develop and practice implementing an Emergency Flood Plan to insure<br>° watches and warnings are quickly received and disseminated to people as well as to public gathering places,<br>° ordinances are in place <u>requiring</u> theaters and other public gathering places to provide warning to patrons,<br>° dangerous road crossings are monitered for closure,<br>° predecisions are made and officials know what to do, and<br>° there is a designated chain of command should any decision maker be unavailable. |
| 7 | Knowledge regarding street flooding and overtopping flows could have saved lives. | Two actions are recommended.<br>° Cities and Counties should provide <u>ongoing</u> public education (schools and media) on what to do in <u>any</u> natural or man-made disaster.<br>° States should have <u>mandatory</u> driver license test questions relating to overtopping floods (and perhaps other travel related disasters as well). |

TABLE 3 (*continued on next page*)

TABLE 3 *(continued)*

| No | Lesson | Recommendations |
|---|---|---|
| 8 | Engineering that fails to recognize the existance of 'super' storms and floods causes deaths and loss of property. | Design engineers and MDP's must<br>° avoid tunnel vision (even as is now occuring at the 100-year design level),<br>° look at worse case scenarios so that there is, as a minimum, an awareness of all probable flood hazards, and<br>° devise affordable solutions to minimize property loss from these hazards and prevent loss of life. |
| 9 | Had solid (Jersey Barrier) or open (box-beam, ARMCO, etc.) traffic safety and control devices been in place where roadway overtopping occurs, property losses would have been much greater and more lives probably lost. Particularly insidious are maintenance contracts to install such devices without a review by hydraulic engineers. | When safety or traffic control devices are required that may protrude into overtopping flood waters,<br>° consider other alternatives (flatten fill slopes in lieu of barriers, lower gradelines, etc.)<br>° remove the property subject to the flood hazard, or<br>° justify causing an increased flood hazard. |
| 10 | Inattention to detail resulted in excessive property damage, costly records damage, curtailed hospital services, and loss of life. | Engineers and regulating agencies must be more attentive to details such as<br>° what occurs when flows are <u>well</u> above curbs,<br>° whether below grade portions of essential buildings adjacent to flood prone streets contain valuable records, costly property, essential services, etc. and if so, whether they are flood-proofed, and<br>° discovering innocent appearing shallow depressions that cause a flood hazard and insuring they are well drained for at least the 100-year flood. |
| 11 | Fencing similar to the chain link type can cause flood hazards. | When this type of fencing must be used in flood prone areas consider<br>° employing rigorous maintenance practices to keep fence free of debris during flood seasons, and<br>° the use of breakaway posts that will fail when floodwaters first begin to accumulate. |
| 12 | Designs accomplished by inexperienced engineers unfamiliar with state of the art practices and standards of practice, as well as lacking a definitive policy or well structured internal corporate organization for decision making will not be defensible in court. | Regulatory agencies and engineering design offices should insure that<br>° knowledgeable, experienced, and licensed hydraulic engineers review drainage designs,<br>° definitive policies with acceptable standards of practice are in place and adhered to, and<br>° the internal organization results in informed, competent drainage related decisions. |
| 13 | Design documentation and past flood documentation increases credibility in court, facilitates preparation of legal defenses, and refutes negligence claims. | Regulatory agencies and engineering design offices should<br>° insure design documentation is complete and available in perpetuity, and<br>° take well focused and coordinated steps to document essential data during and immediately following a flood. |

that time, as in the Cheyenne flood, the courts will probably award damages, and dictate specific and unnecessarily costly corrective measures. In short, highway agencies or communities have a choice of paying now, or paying an exorbitant amount later—while abetting death and misery.

Following the Cheyenne flood, the mayor of Cheyenne reappointed the aforementioned task force of flood professionals and community experts to identify how best to eliminate Cheyenne's flood hazards. The actions of the task force have resulted in the completion of an MDP for Cheyenne. The city is now attempting to develop a reliable funding source. A drainage utility appears to be the best alternative. Perhaps not surprising is the apparent lack of strong public support to provide a reliable funding source—with a downturn in the local economy plus five flood-free years, along with a turnover in population, people tend to forget.

The Cheyenne flood did reveal some flood hazard areas that could be quickly addressed without waiting for the findings of an MDP. The city has moved as quickly as practicable to correct these problems by enlarging some drainage structures, constructing a major storm drain, moving the ECC, and implementing a flood early-warning system. However, if Cheyenne's flood hazards are to be reduced to acceptable levels, much work remains.

## LESSONS AND RECOMMENDATIONS

The foregoing discussion has been pointing towards enumerating specific lessons learned from a flash flood, as well as providing specific recommendations for corrective actions. These lessons and recommendations may be community, agency, and site specific, but each should still be given careful consideration. Possibly some defendant's attorney may drag this report out of a file some day and question a responsible party about practices in light of the documented 13 lessons and recommendations provided in Table 3.

## CONCLUSION

In less than 2 hr in which 12 lives and $65 million were lost and much misery was created, Cheyenne, Wyoming, learned some painful lessons about flash floods. These lessons (and recommendations) are presented in Table 3 in the hope that other highway agencies and communities can learn from this experience. Although implementing the recommendations would be costly, failure to do so may be much more costly—and painful.

## REFERENCES

1. R. D. Jarrett and J. D. Costas. *Evaluation of the Hydrology in the Colorado Front Range Using Precipitation, Streamflow, and Paleoflood Data for the Big Thompson River Basin.* U.S. Geological Survey, Water-Resources Investigations Report 87-4117, Denver, Col., 1988.
2. S. A. Druse, M. E. Cooley, S. L. Green, and H. W. Lowham. *Flood of August 1, 1985, Cheyenne, Wyoming.* Hydrologic Investigations Atlas, U.S. Geological Survey and Wyoming State Highway Department, 1986.
3. C. F. Chappell and D. M. Rodgers. *Meteorological Analysis of the Cheyenne, Wyoming, Flash Flood and Hailstorm of 1 August 1985.* U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Research Laboratories, Dec. 1988.
4. P. T. Tyrrell. *Development of Design Rainfall Distributions for the State of Wyoming.* Thesis, Department of Civil Engineering, University of Wyoming, Laramie, Dec. 1982.
5. J. B. Linder-Lunsford. *Precipitation Records and Flood-Producing Storms in Cheyenne, Wyoming.* Water-Resources Investigations Report 87-4225, U.S. Geological Survey and city of Cheyenne, Cheyenne, Wyo., 1988.

*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# Shoreline Stabilization To Protect Louisiana Highway 82

BABAK NAGHAVI AND PETER A. ALLAIN

Several attempts by the Louisiana Department of Transportation and Development to protect the shoreline and Louisiana Highway 82 are discussed. To identify the important factors needed for proper design of shore protection structures, failures of earlier efforts, as well as the success of recent efforts, are evaluated. An emphasis should be placed on the acquisition of wave data, as well as a survey of the offshore profile. Porosity of the revetments, underlayers, and geotextiles seems to be important in the stability of the structure. Scour at the toe is another design consideration in design of revetment structures. Project cost of shoreline protection was considerably reduced by providing alternates through competitive bidding. The 1984 and 1988 cabled and noncabled blocks and cast-in-place revetments have been exposed to several storms and have performed satisfactorily. The only damage to the roadway has been associated with landside foreslope at the time of overtopping.

Shoreline stabilization to stop erosion is an uncommon problem for most state transportation agencies. Occasionally, however, such corrective measures are needed to protect highways threatened by erosion in coastal areas. Identification and elimination of the cause of erosion is normally the best approach for solving the problem, but cannot always be accomplished. The second-best approach for solving the problem is the construction of coastal structures (*1*). Of these structures, revetments for shore stabilization is the most common. In order to avoid adverse results, revetment projects should be designed cautiously, and may require supplemental measures such as breakwaters or beach nourishment (*2*).

Over 36 mi of seawalls and shoreline revetment are located in Louisiana, primarily along Lake Pontchartrain and East Timbalier Island (*3*). Louisiana has experienced accelerated coastal erosion in recent years. One of the few coastal areas in the state directly accessible by car is located along Louisiana Highway 82 (LA-82) in Cameron Parish. Between Peveto Beach and Holly Beach, LA-82 roughly parallels the Gulf of Mexico shoreline. Along this section of highway, the roadway embankment foreslope is under frequent wave attacks. Storm occurrences and their associated stages measured inland at the U.S. Coast Guard station are presented in Table 1. Initial efforts by the Louisiana Department of Transportation and Development to protect the shoreline and LA-82 were unsuccessful. However, recent efforts have been successful, to date. Figure 1a shows the failure of the initial shoreline protection efforts; Figure 1b shows one of several recent revetment projects along LA-82. The major goal for these projects was to reduce the repair costs associated with frequent storms, rather than to protect against major hurricanes. The experience gained during the design and construction of shore protection projects along LA-82 is described.

B. Naghavi, Louisiana Transportation Research Center, 4101 Gourrier Avenue, Baton Rouge, La. 70808. P. A. Allain, Department of Transportation and Development, P.O. Box 94245, Baton Rouge, La. 70804–9245.

## AREA DESCRIPTION

The project site is located in Cameron Parish, in extreme southwestern Louisiana. LA-82, an asphalt two-lane road, is the only east-west road in this area. Figure 2 shows the general area of concern, which lies between Constance Beach and Holly Beach. LA-82 roughly parallels the shoreline in this area, with the width of the sandy beach (roadway toe of slope to shoreline) varying from 10 to 30 ft. The most critical area is a 4-mi section beginning at and to the west of Constance Beach, where the roadway embankment foreslope is under frequent wave attacks. Communities served by this highway include Johnson's Bayou, Ocean View Beach, Constance Beach, and Holly Beach. This highway serves as a hurricane evacuation route, affording residents of Johnson's Bayou, Ocean View Beach, and Constance Beach egress from west to east, and thence northward along LA-27. Located slightly landward and parallel to the highway are several large pipelines and the Sabine National Wildlife Refuge. The existence of the beach between the highway and the gulf also provides an attraction for weekend and summer tourists.

## GEOMORPHIC SETTING AND COASTAL PROCESSES

Beaches in this area lie in the chenier plain of southwestern Louisiana. Cheniers are geologic formations similar to beach ridges (*4*). Natural beach ridges are composed of shells, shell fragments, silt, and sand. Beaches vary in thickness from 2 to 20 ft, and in width from 100 to 1,500 ft (*5*). The chenier forms the substrate for LA-82. The highway cannot be economically relocated any further inland because brackish and saline marshes occur landward of the chenier. The long-term shoreline retreat rate for the vicinity of the project area has been approximately 4 ft/year from 1883 to 1968 (*5*). However, local erosion rates are often in excess of 15 ft/year during and after major storms or hurricanes (*4*).

Wave energy typically is low; breaking wave heights average 2 ft, with wave periods of 5 sec. The prevailing direction of wave approach is from the southeast, which results in predominantly westward littoral transport. The average rate of transport varies from about 62,000 to 100,000 yd$^3$ per year

TABLE 1 STORMS AND THE RECORDED STAGES (1940 TO 1989)

| DATE OF STORM | STORM NAME* | STAGE** |
|---|---|---|
| 1940 Aug 2-10 | H | |
| 1940 Sept 19-24 | TS | 3.5 |
| 1941 Sept 11-17 | TS | 5.2 |
| 1942 Aug 17-22 | H | 1.5 |
| 1943 July 25-29 | H | 4.0 |
| 1943 Sept 15-19 | TS | 3.4 |
| 1946 June 13-16 | TS | 3.2 |
| 1947 Sept 4-21 | H | 2.84 |
| 1949 Sept 27-Oct 6 | H | 3.87 |
| 1954 July 27-30 | TS Barbara | 2.7 |
| 1956 June 12-14 | TS | no record |
| 1957 June 25-28 | H Audrey | 12.90 |
| 1957 Aug 8-11 | TS Bertha | no record |
| 1957 Sept 16-19 | TS Esther | no record |
| 1961 Sept 3-15 | H Carla | 7.27 |
| 1963 Sept 16-19 | H Cindy | 3.72 |
| 1964 Sept 28-Oct 5 | H Hilda | 3.76 |
| 1965 Aug 26-Sept 12 | H Betsy | 4.78 |
| 1969 Feb 14 | Storm | 5.05 |
| 1970 Feb 1 | Storm | 4.65 |
| 1970 Aug 3 | TS Celia | 4.2 |
| 1971 Sept 3-13 | H Edith | 5.09 |
| 1972 Oct 16 | Storm | 5.50 |
| 1973 Sept 5 | TS Delia | 5.43 |
| 1974 Aug 29-Sept 10 | H Carmen | 3.4 |
| 1977 Sept 1 | H Anita | 4.57 |
| 1978 Aug 26-29 | TS Debra | no record |
| 1979 July 24 | Storm | 5.50 |
| 1979 Sept 20 | TS Claudette | 5.33 |
| 1980 May 19 | Storm | 5.57 |
| 1982 Sept 9-12 | TS Chris | 4.20 |
| 1983 Aug 15-21 | H Alica | 4.51 |
| 1985 Aug 12-20 | H Danny | 3.62 |
| 1985 Oct 26-Nov 1 | H Juan | 3.70 |
| 1986 June 23-28 | H Bonnie | 4.75 |
| 1988 April 30 | Storm | 3.97 |
| 1988 Sept 12-16 | H. Gilbert | 4.40 |
| 1989 August 5-6 | H. Chantal | 5.10 |
| 1989 October 15 | H. Jerry | 4.2 |

* TS Tropical Storm
H Hurricane

** Gage located inland at Coast-Guard Station, Cameron, La., maintained by the U.S. Army Corps of Engineers. Gage zero is at Mean Low Gulf (MLG to MSL subtract 0.78').

(5). The Calcasieu River is a major source of suspended sediments for the immediate Holly Beach area. However, sediments supplied by the Calcasieu River do not contain sufficient granular material to maintain the existing beach. The movement of the sediments is also influenced by a navigational jetty located at the mouth of the Calcasieu River. The prevailing east-to-west littoral currents are eroding the materials from the immediate vicinity of Holly Beach, and depositing them on the downdrift side, west of Ocean View Beach.

Tides at Holly Beach are diurnal, with a daily range of about 2.0 ft. Mean daily tide level is 1.0 ft above the mean low gulf (MLG). Abnormally high tides occur during hurricanes; the maximum high tide of over 12 ft was recorded during hurricane Audrey in 1957. High storm tides allow higher waves than usual to attack the shoreline because of greater water depths. Major erosion damages to the project area have been caused by such wave actions.

## HISTORY AND PREVIOUS SHORE PROTECTION EFFORTS

Before 1932, the roadway between Cameron and Johnson's Bayou serviced only local traffic. When a ferry was installed in 1932 at the Sabine River, the coastal roadway became an important link between the coastal communities of Texas and Louisiana. In 1936, the roadway was made into a state route and was resurfaced with shell. The highway was resurfaced with shell again in 1945 and 1951. The original highway was located on a low sand ridge that was once the back side of a

FIGURE 1 *Left*: A section of LA-82 after tropical storm Chris in 1982, and *Right*: one of the several revetment projects along LA-82.

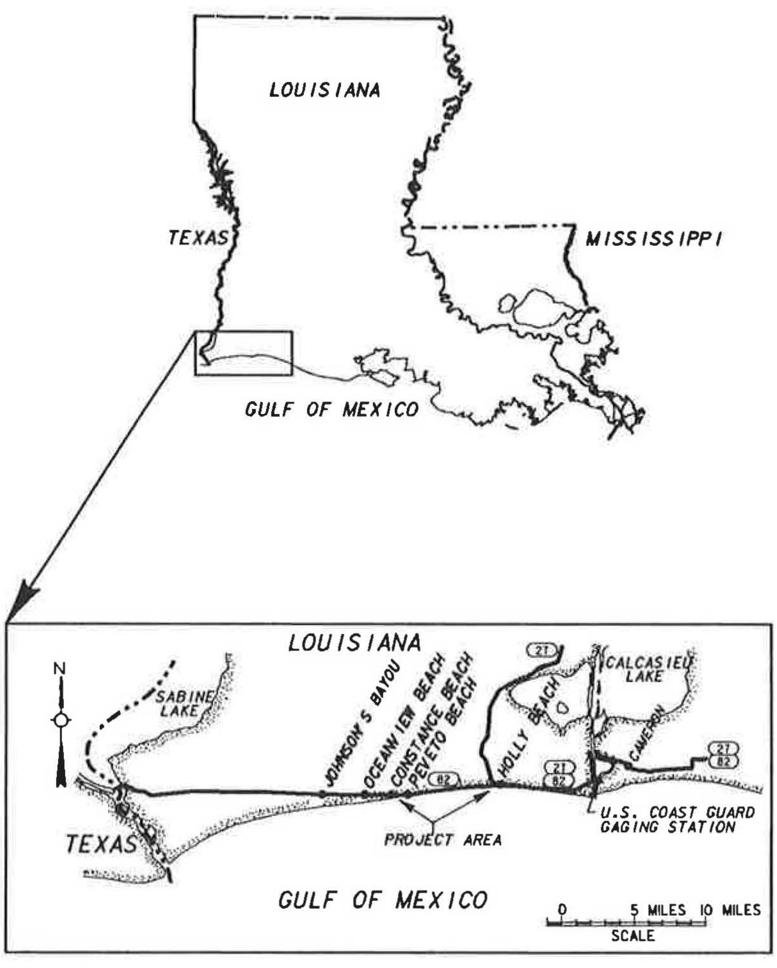



FIGURE 2 Vicinity map.

chenier. At this location, the highway was well protected from the gulf by at least 300 ft of beach and dunes. The highway became increasingly exposed to wave damage as the beach diminished and had to be relocated inland several times. Figure 3 shows the regression of the shoreline and the corresponding relocation of the highway. As further relocation of the highway became unsuitable, various revetment projects were initiated by the Louisiana Department of Transportation and Development. The project limits for these projects are shown in Figure 4.

The loss of beach was accelerated by several major storms, which in turn made the highway vulnerable to subsequent storms. In June 1957, hurricane Audrey was the most severe storm of record to hit the area. The storm caused major damage to the highway and destroyed all the residences at Holly Beach, as well as most of the residences at the nearby town of Cameron. Erosion caused by hurricane Audrey left a 3-mi section of the highway exposed to wave action from the gulf.

LA-82 was greatly improved when the asphalt surfacing of the roadway was completed in May 1960 at a cost of $429,000. In September 1961, hurricane Carla caused severe damage to the newly completed highway. As a result of hurricane Carla, the 3-mi section of the roadway embankment exposed by hurricane Audrey was destroyed. The roadway was reconstructed 30 ft inland. Repairs from hurricane Carla were completed in April 1963 at a cost of $308,000.

In September 1963, hurricane Cindy damaged 1½ mi of the reconstructed roadway. From 1964 to 1969, the beach fronting the road continued to erode. The loss of beach left a 4-ft-high escarpment at the edge of the shoulder that required continual maintenance to prevent undermining of the pavement.

As the beach continued to erode and no other suitable location could be found to relocate the highway, state officials decided to protect the roadway. The first roadway protection project consisted of revetting the gulf-side roadway embankment foreslope. After considering several alternatives, the state engineers decided to use a 4-in.-thick cellular concrete block revetment. This 4-in.-thick block, shown in Figure 5 and labeled Type A block, was developed in Holland. In order to determine the stability of these blocks, a 1:5 scale model test was conducted at Voorst Laboratory, a part of the Delft Laboratory in Holland. The scale model test, completed in December 1968, indicated that the revetment would remain stable for wave heights up to 3.6 ft. In order to test the prototype for real situations, a 200-ft prototype revetment was built along LA-82 by the Louisiana Department of Transportation and Development in January 1969 (*6*). The revetment

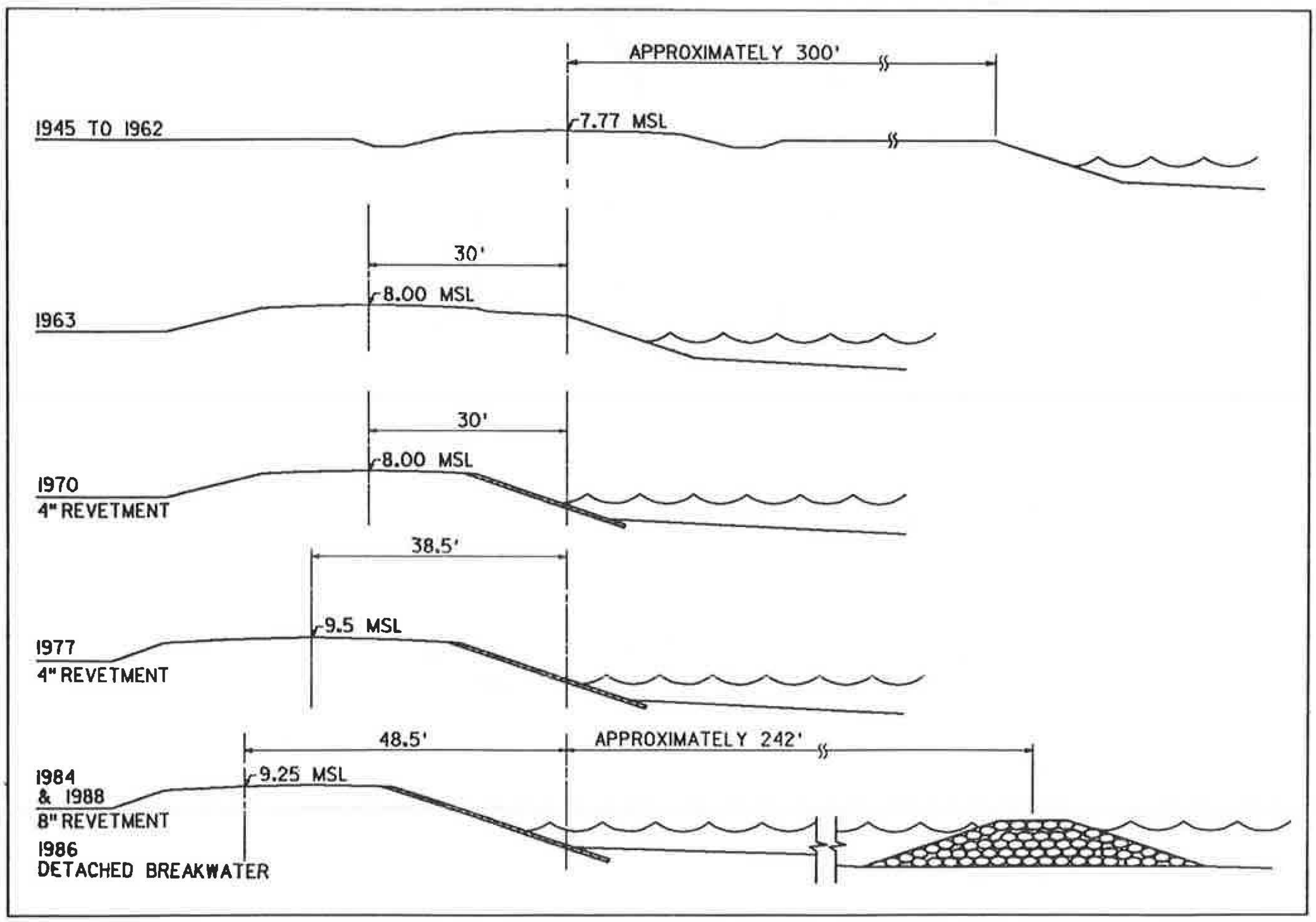



**FIGURE 3 Regression of shoreline and corresponding relocation of LA-82.**

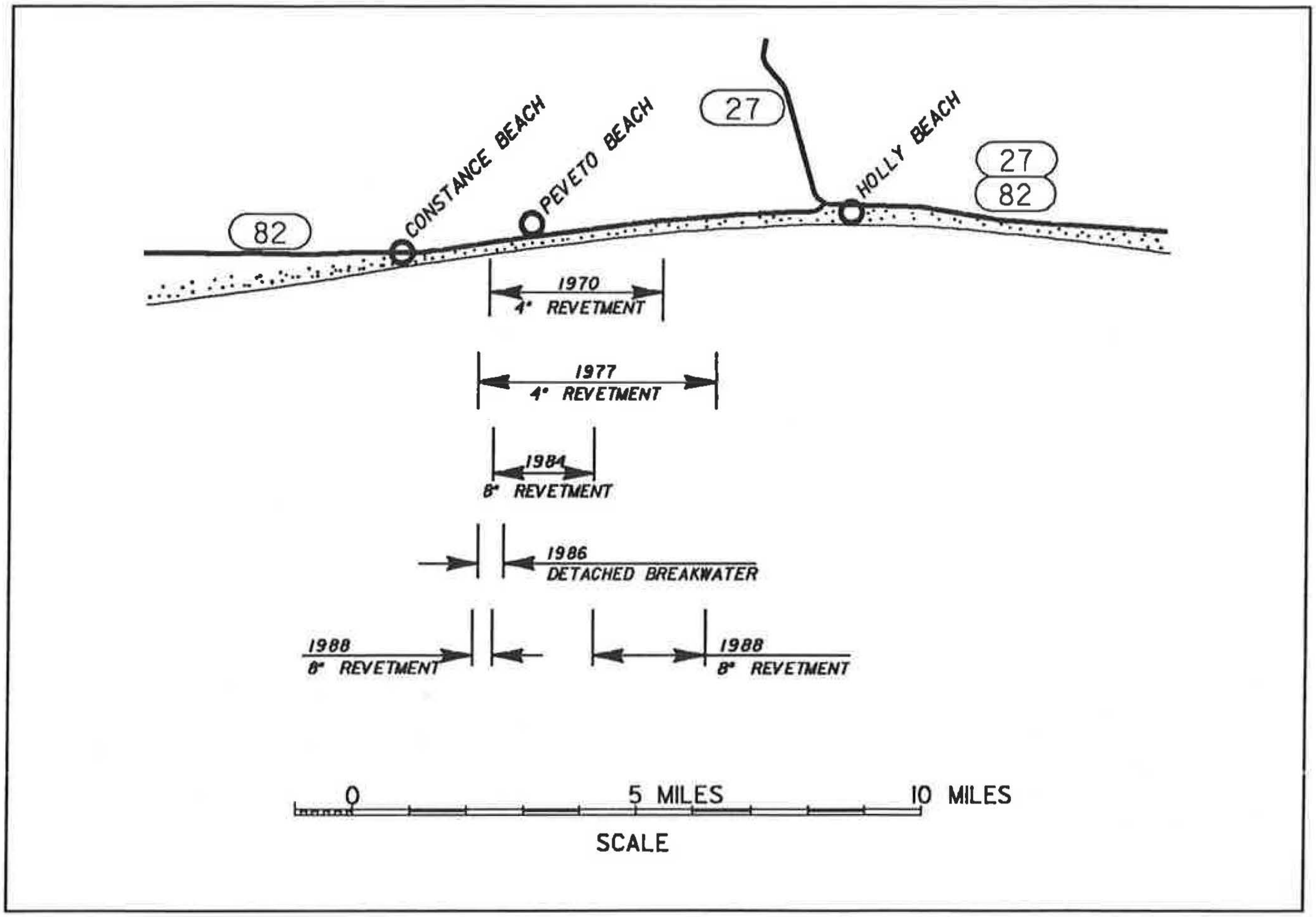
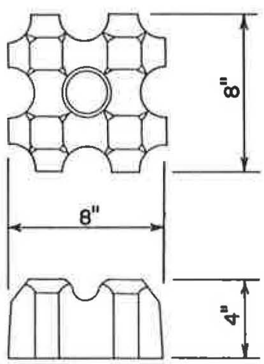



**FIGURE 4 Limits of coastal erosion protection projects.**



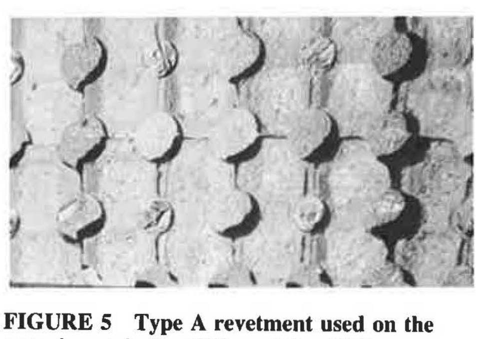



**FIGURE 5 Type A revetment used on the experimental, the 1970, and the 1977 revetment projects.**

was divided into two test sections, each being 100 ft long, to evaluate the underlying geotextiles. The 100-ft sections were underlain either by low- or high-porosity geotextiles.

The experimental revetment site was put through a severe test soon after the project was completed. On February 14, 1969, an unusual winter storm brought high tides to an elevation of approximately 5 ft MLG (recorded at Cameron Coast Guard Station, located 10 mi to the east), with accompanying waves estimated at 3 to 4 ft in height. The concrete block revetment that was placed on the low-porosity geotextile was completely destroyed. The concrete block revetment placed on the high-porosity geotextile remained essentially intact, with 65 ft of the original 100 ft of construction withstanding the storm. The failure of the 35-ft section with the high-porosity geotextile was attributed mainly to flanking and failure at the adjacent test section (that with the low-porosity geotextile). This storm destroyed the entire gulf-side lane of the highway for 1 mi on each side of the test section. The repairs to the roadway were completed in April of 1969 at a cost of $73,144. Another unusual winter storm of relatively smaller magnitude occurred at the test site on February 1, 1970, less than 1 year after the first storm. The remaining concrete block revetment test section survived this storm with only minor damage at the flanks. This storm caused major damage to 1.5 mi of the highway. The two winter storms of February 1969 and February 1970 were estimated to be events of 15- and 12-year frequency, respectively. Therefore, considering the results of scale model tests at Delft Hydraulics Laboratory and test installations at Holly Beach, it was concluded that cellular concrete block revetment used in conjunction with high-porosity geotextiles and adequate toe and flank would prevent major damage to the road for the 25-year event.

Plans for a 3-mi beach protection project using the results from the test project were prepared and the construction was completed in December 1970. The in-place cost of cellular concrete block revetment was $8.5 per square yard, for a total project cost of $398,887. From the time this revetment was completed in 1970 until September of 1973, the highway in this area was inundated approximately 20 times. Only minor damage occurred that required only minor repairs. However, tropical storm Delia in September 1973 caused major damages to a 0.25-mi section of the revetment and roadway. Repairs were made to the roadway at a cost of $121,647. The revetment was not repaired until some time later, when additional damage required action.

To completely repair the storm damage from tropical storm Delia and to reduce the number of times that the highway would be inundated during storms, a project to raise approximately 4 mi of LA-82 (in the concerned area) was initiated in 1974 and completed in March 1977. In this project, the roadway elevation was raised 1.5 ft to extend the length of time the highway would be passable as a storm or hurricane approached. The project also required relocating the centerline 8.5 ft inland, and extending the revetment upslope. To protect the granular embankment fill placed over the old roadway, the inland highway foreslope was paved with soil cement. To protect the bottom edge of the revetment from scour, rock placed on the geotextile was constructed to form a revetment toe. The total cost of this project was $4,192,282, most of which was associated with the raising of the roadway.

## RECENT SHORE PROTECTION EFFORTS

From 1977 to 1983, the foreslope of the revetted roadway was ravaged by destructive forces of numerous hurricanes and tropical storms. In 1979, tropical storm Claudette damaged the revetment and paved shoulders. Additional damage caused by several other storms necessitated continual maintenance to the roadway and revetment. In March 1984, another experimental revetment project was completed. The objective of this project was to protect LA-82 from further erosion. The centerline of the roadway was offset another 10 ft (landward) for the destroyed roadway section at the time of this project. By this time, about 1.7 mi of the 3 mi of 4-in.-thick Type A block revetment installed in 1970 and 1977 had been completely destroyed, as well as the gulf-side lane of the roadway. The 1.7-mi revetment portion of the project specified alternating sections of five revetment types, with a minimum thickness of 8 in. Of the five, four revetment types were precast cellular blocks, and one was the cast-in-place type. Figures 6–9 show the four precast, cellular block revetments, which are labeled Type B, C, D, and E blocks. The total cost of this revetment project was about $3.24 million. This project has performed satisfactorily to date, and has been able to stop the beach erosion from further advancement.

In January 1986, six segmented breakwaters were constructed to protect the revetment and the highway, at a total cost of $423,000. The use of different designs also provided the opportunity to evaluate the performance of each breakwater and to identify the shoreline responses. The use of detached breakwaters to dissipate most of the wave energy through turbulence has been successfully implemented in Japan and other countries. However, their use as a coastal erosion control tool requires a detailed study of the waves, currents, sediment supply, and sediment transport in the region (*7*). The design and performance of the 1986 detached breakwaters are discussed in detail by Nakashima et al. (*4*). Even though the detached breakwaters provided additional protection, the relatively short length of the breakwaters, as shown in Figure 4, had a minimal effect on the overall performance of the recent revetment projects.

The success of the 1984 revetment project and the continuing failure of the light revetment installed in the previous projects brought about another revetment project, which was completed in August 1988. The typical section used for this project is shown in Figure 10. In this project, an additional 2.3 mi of the beach along LA-82 was protected with two classes of revetment. The first class of revetment was selected from the five types previously used in the 1984 revetment project. In order to reduce cost through competitive bidding, the plans for this class of revetment allowed the contractor a choice between Types C and D cabled blocks. The contractor chose and installed Type C blocks at approximately half the cost of the original 1984 project installation cost. The second class of revetment was an experimental, cast-in-place, cellu-

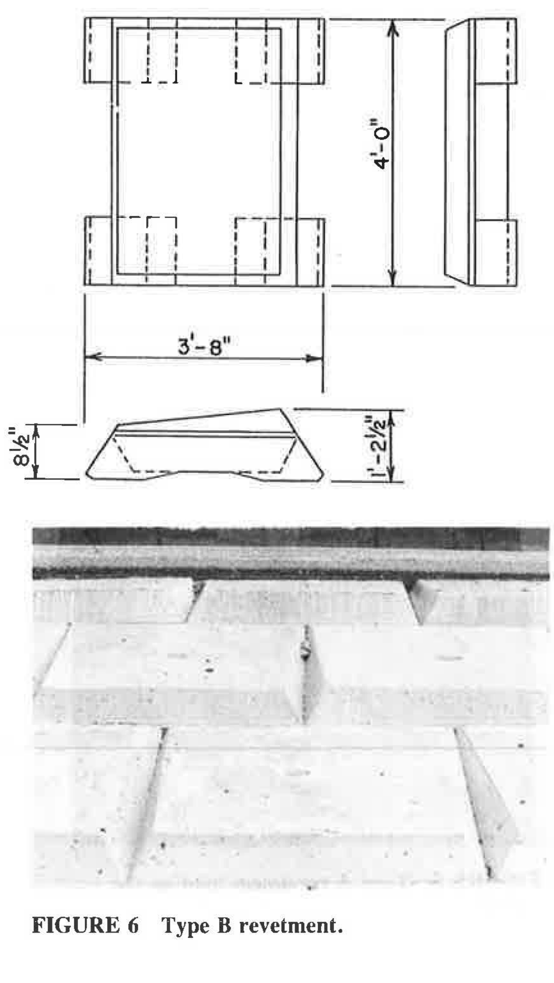



**FIGURE 6 Type B revetment.**

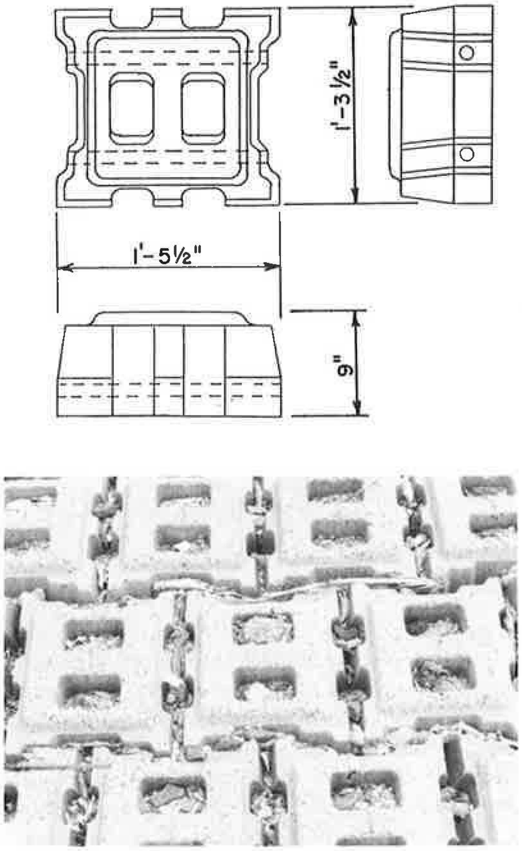



**FIGURE 7 Type C revetment.**

lar, concrete block revetment. Again, to reduce costs through competitive bidding, similar blocks from two manufacturers were allowed in the project plans. The contractor selected the Type F block, which is shown in Figure 11. The total cost of this project was $2.37 million. While under construction, sections of the project were damaged by a severe storm on April 30, 1988, that overtopped the roadway. This storm damaged completed and uncompleted sections of the newly installed precast cellular blocks, but the revetment installed in 1984 sustained no damage. The damage to the newly installed, completed revetment sections is believed to be caused by slope instability. Figure 12 shows the revetment displacement caused by slope instability. In addition to revetment displacement, voids were formed at several locations beneath the revetment. The increased cost to the project because of storm damage was $250,000. Repairs included filling the depression in the revetment with concrete and pumping grout into the voids beneath the revetment. The completed project was then subjected to severe weather again in 1988 by hurricane Gilbert. Although hurricane Gilbert did not cross the Louisiana coast, the associated violent weather forced the closing of LA-82 because of the extreme tides and waves. The roadway and revetment were overtopped, but were not severely damaged.

On August 5, 1989, hurricane Chantal crossed land just west of the Texas-Louisiana border, causing severe damage to LA-82. Although the revetment was only slightly damaged, sections of the landside shoulder and adjacent travel lane were heavily damaged by overtopping waves. As the waves overtopped the revetment and the roadway, the landside shoulder was eroded. With the loss of the shoulder, the embankment under the landside travel lane was scoured out, causing sections of the pavement to collapse. The soil-cemented, landside foreslope installed as part of the 1977 4-in.-thick revetment project did not adequately protect the roadway embankment. Repairs to the shoulder, embankment, and roadway are estimated to cost $250,000. Similar damage to the roadway also occurred during hurricane Jerry in October 1989.

Although maintenance of all roadway surfaces is required periodically, LA-82 is unusual because of repeated damage caused by storms in the Gulf of Mexico. In addition to normal roadway maintenance, additional expenditures for roadway relocation, revetments, and breakwaters have been required. Table 2 presents the expenditures that the state has made on LA-82. With the exception of hurricane Chantal in 1989, the roadway has not been seriously damaged at the sites of 1984 and 1988 revetment projects.

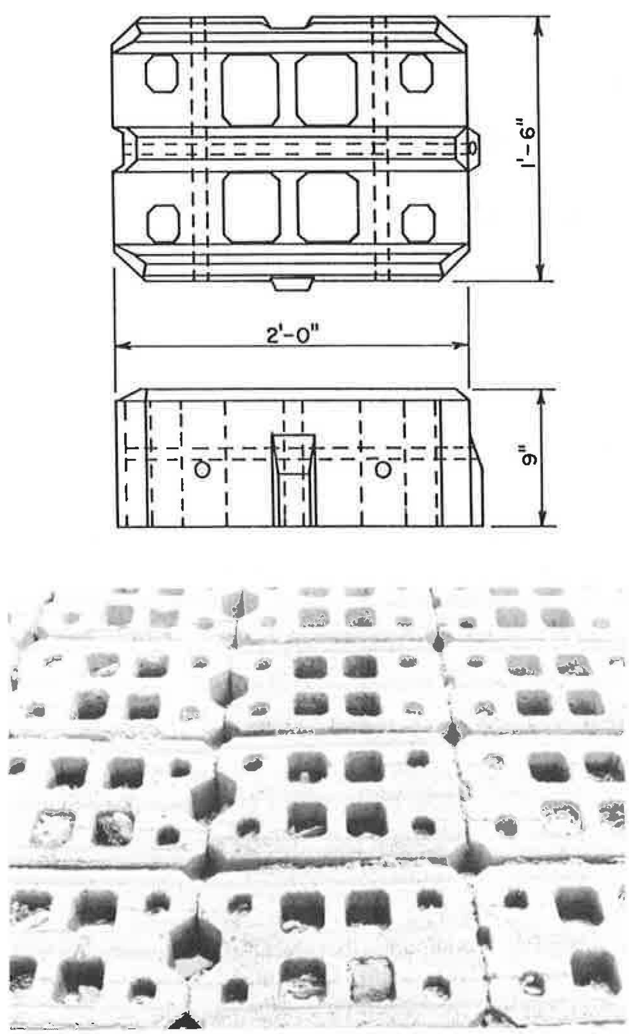



**FIGURE 8 Type D revetment.**

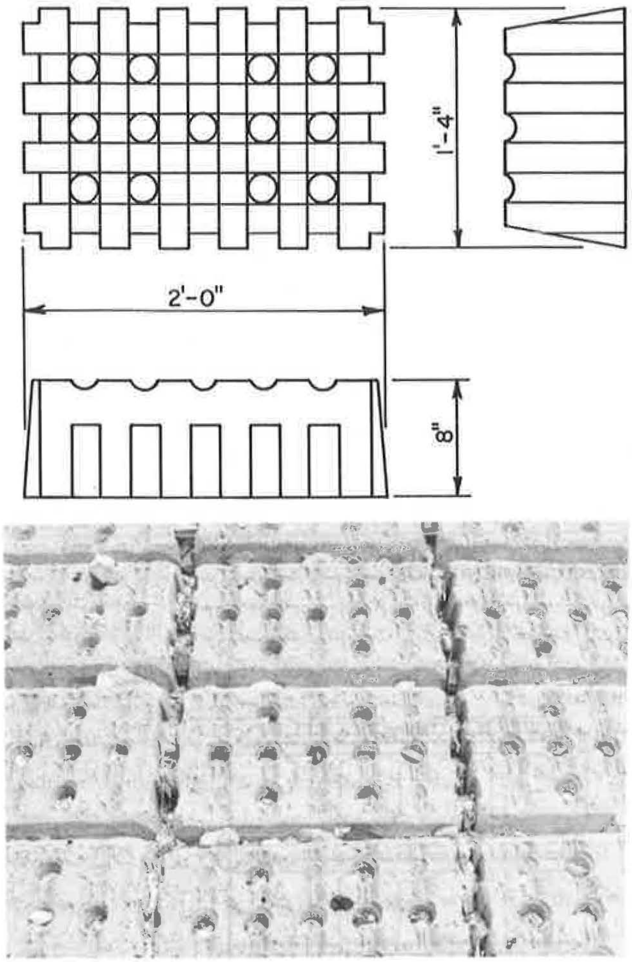



FIGURE 9 Type E revetment.

to occur for the design storm (*9*). The existing roadway elevation, and therefore the elevation of the top of the revetment, was established for LA-82 in 1977, when the road grade was raised to increase the time for which the highway would be passable during evacuations. The majority of the storm damage that has occurred to the highway has been to the landside shoulder. The landside shoulder becomes severely eroded when waves wash over the top of the revetment and roadway. When the damage to the shoulder was severe enough, as with hurricane Chantal in August 1989, embankment material and road pavement were severely damaged. It is believed that the soil cement installed in 1977 to protect the embankment failed because of the hydrostatic pressure. Delft Hydraulic Laboratory had recommended that the landside shoulder be constructed with a permeable structure to relieve the excess pressure and to provide drainage from the wave run-up (*10*).

### Beach Characteristics

An offshore profile survey is essential for proper design of revetments. Beach slopes are subjected to forces of breaking waves, wave uprush, and downrush. Wave energy is dissipated by roughness and turbulence of the flow propagating on the slope. The beach generally tends to adopt or develop the most stable profile with respect to wave action and the physical properties of the soil. Therefore, careful consideration should be given to design of revetment projects when the slope is cut to reduce the quantity of the revetment or because of construction limits. The importance of slope stability was demonstrated by the displacement of portions of the 1988 newly constructed revetment, as shown in Figure 12. When the slope has to be modified, specifically as a fill, the construction should benefit from circumstances like tides and other natural forces to shape the profile of its final design limit by allowing for the settlement and proper soil density. If the revetment is placed on fill, the fill must be suitably compacted (*9*). Settling is mostly the result of poor compaction of the subsoil or fill in revetment.

## DESIGN CONSIDERATIONS

Design of a successful beach protection project requires a full understanding of the natural forces involved. Most failures occur when the designer overlooks the balance that must be maintained between these forces. In design of revetment projects, the following factors should be considered.

### Wave Data

The wave data should be analyzed carefully. Waves are the major factor in determining the geometry and composition of beaches. Other hydraulic properties of the beach, such as water level fluctuations during high and low tides and recorded historical stages during past storms, are also essential parts of the design. The U.S. Army Corps of Engineers Shore Protection Manual (*8*) contains the procedures for analyzing wave data. Wave condition during the highest possible tide is important, because this condition produces maximum wave attack on the revetment. The height of the structure should be at least equal to the elevation of maximum uprush expected

### Revetment Design

Departure of concrete block from the slope because of being too light is the main reason suspected for the failure of the 4-in.-thick Type A revetments installed in 1970 and 1977. The larger 8-in.-thick Types B, C, D, and E revetment blocks installed in 1984 provided far more weight. As indicated in Table 3, these larger blocks provided at least 67 lb/ft$^2$, in comparison to the 29 lb/ft$^2$ of the 4-in.-thick revetment. To further reduce the chance of loss of individual blocks, only cabled revetments were considered for the 1988 8-in.-thick revetment project. Corrosion-resistant polyester cables were used to tie the blocks. The use of heavy blocks that are cabled together and anchored with an anchoring mechanism at the top of the mat should provide significant stability, and therefore reduce the chance of failures. Tables 3 and 4 present the physical characteristics and the amounts installed and cost, respectively, of the individual revetments.

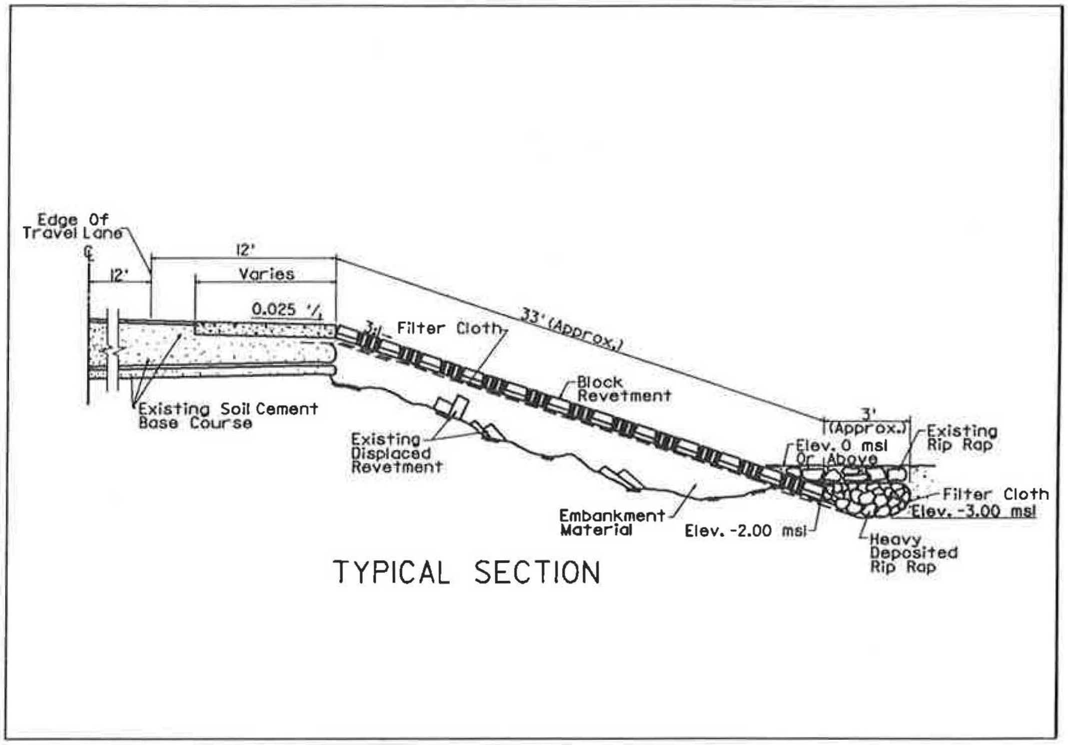



**FIGURE 10 Typical section for 1988 8-in. revetment project.**

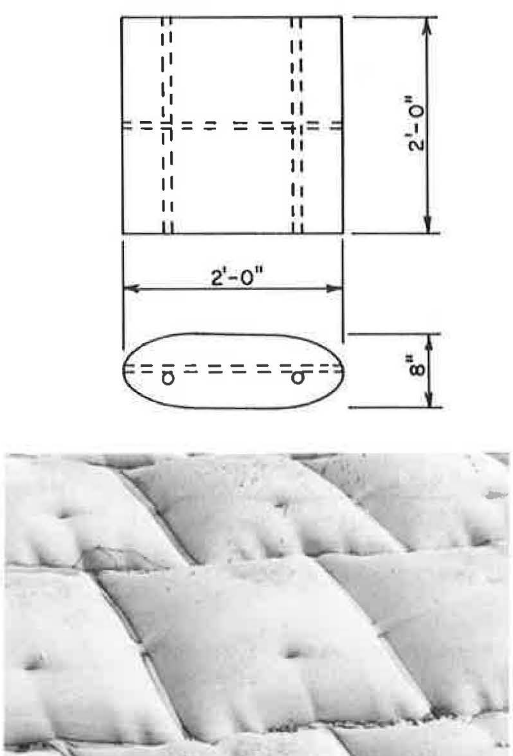



**FIGURE 11 Type F cast-in-place cellular concrete revetment.**

## Permeability

An important factor in the stability of revetment is the porosities of the revetments, underlayers, and geotextile used. The opening of the geotextile should be small enough to retain the sand, but porous enough to provide adequate relief of hydrostatic pressure. The importance of adequate porosity of the geotextile was demonstrated by the 1970 test projects. The evaluation and assessment of the test sections resulted in the conclusion that because of inadequate drainage, excess

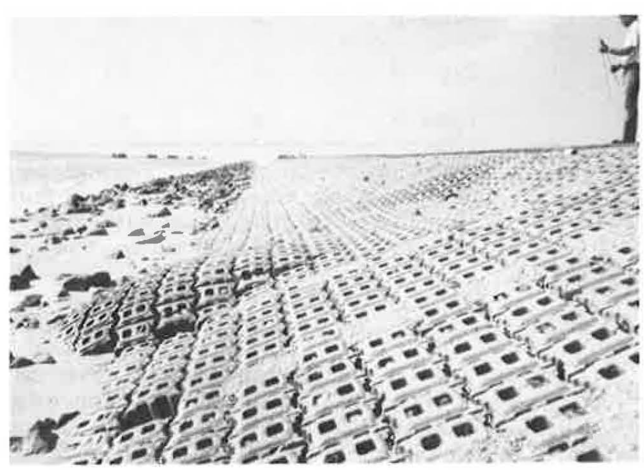


**FIGURE 12 Revetment displacement caused by slope instability.**

TABLE 2 SUMMARY OF EXPENDITURES ON LA-82 FROM 1945 TO 1989

| Completion Date | Project Description* | Cost |
|---|---|---|
| 1945 Nov | Shell surface | $ 51,893 |
| 1951 Feb | Shell surface | $ 94,650 |
| 1960 May | Asphalt surface | $ 922,216 |
| 1963 April | H Carla repairs | $ 308,000 |
| 1969 April | Storm repairs | $ 73,144 |
| 1970 Dec | Revetment project | $ 398,887 |
| 1971 Aug | H Celia repairs | $ 18,917 |
| 1971 Aug | Storm repairs | $ 10,303 |
| 1975 Nov | TS Delia repairs | $ 121,647 |
| 1977 April | Revetment & raise road | $ 4,119,905 |
| 1982 Sept | TS Chris repairs | $ 4,157 |
| 1984 Mar | Revetment project | $ 3,240,939 |
| 1984 Dec | Storm Repairs | $ 21,789 |
| 1985 Nov | Asphalt overlay | $ 98,702 |
| 1986 Jan | Breakwater project | $ 423,321 |
| 1986 July | H Bonnie repairs | $ 72,815 |
| 1988 April | Storm repairs | $ 13,005 |
| 1988 Sept | H Gilbert repairs | $ 12,825 |
| 1988 Oct | Revetment project | $ 2,365,164 |
| 1989 Aug | H Chantal | $ 250,000 |
| 1989 Oct | H Jerry | $ 60,000 |
| Total Cost 1945 - 1989: | | $12,682,279 |

* TS Tropical Storm
H Hurricane

TABLE 3 REVETMENT CHARACTERISTICS

| Type | Thickness (inches) | % Open Area | Wt/Unit (lbs/each) | Wt/ft$^2$ (lbs/ft$^2$) | Reinforced or Cabled |
|---|---|---|---|---|---|
| Type A | 4 | 30 | 13 | 29 | No |
| Cast In Place | 12 | 0 | N/A | 150 | Yes[1] |
| Type B | 8½ | 4 | 1600 | 133 | No |
| Type C | 9 | 20 | 130 | 73 | Yes[2] |
| Type D | 9 | 20 | 230 | 78 | Yes[3] |
| Type E | 8 | 17 | 150 | 67 | No |
| Type F | 8 | 10 | 373 | 86 | Yes[4] |

[1]The Cast-in-place revetment is reinforced with Grade 40 reinforcing steel with 5/8-inch diameter bars at 5-inch centers parallel to the roadway and 1/2-inch diameter bars at 23-inch centers perpendicular to the roadway.

[2]The Type C revetment system is tied together with 5/16-inch polyester cables at 8-inch centers perpendicular to the roadway.

[3]The Type D revetment system is tied together with 1/2-inch polyester cables at 18-inch centers parallel to the roadway and at 12-inch centers perpendicular to the roadway.

[4]The Type F revetment system is tied together with 5/16-inch polyester cables at 24-inch centers parallel to the roadway and at 12-inch centers perpendicular to the roadway.

TABLE 4 DATE COMPLETED, AMOUNT INSTALLED, AND COST PER SQUARE YARD OF REVETMENT USED TO DATE TO PROTECT LA-82

| Type | Year Installed | Amount ($yd^2$) | Cost ($\$/yd^2$) | Length (feet) |
|---|---|---|---|---|
| Type A | 1970 | 42,133 | 8.50 | 15,800 |
| | 1977 | 25,750 | | 2,714 |
| | | Total 4" Revetment | | 18,514 |
| Cast-in-place Concrete | 1984 | 8,803 | 82.17 | 2,401 |
| Type B | 1984 | 6,336 | 86.17 | 1,728 |
| Type C | 1984 | 14,439 | 77.17 | 3,938 |
| | 1988 | 37,400 | 40.55 | 10,200 |
| Type D | 1984 | 5,694 | 66.17 | 1,553 |
| Type E | 1984 | 6,296 | 66.17 | 1,717 |
| Type F | 1988 | 6,688 | 28.95 | 1,824 |
| | | Total 8" Revetment | | 23,361 |

hydrostatic pressure was created in the slope, resulting in displacement of the underlying sand as well as the revetment. The revetments and underlying geotextiles installed in the 1984 and 1988 projects provided adequate porosity. The experience at Holly Beach also proved that the geotextile with 300-μm openings had a high retention for the fine sand (*10*). The $D_{50}$ size of 150 μm was reported in the U.S. Army Corps of Engineers Report (*5*). However, the $D_{50}$ size for the five samples (of 0- to 1-ft depth) collected and analyzed in 1989 ranged from 160 to 285 μm, with an average of about 205 μm. The Geotextile Engineering Manual (*11*) prepared for the FHWA contains a comprehensive guide for the design and selection of geotextiles. Another cause of potential failure is associated with the deterioration of the geotextile because of adverse environmental conditions, including ultraviolet exposure, abrasion, etc. Table 5 presents the properties of the geotextiles used for revetment projects along LA-82. Contrary to previous indications that a high degree of permeability is required for a stable revetment, the cast-in-place concrete revetment installed in 1984 has performed without failure, probably because the revetment is heavy enough to resist the hydrostatic pressure.

### Scour

Scour at the toe of the revetment structure is another reason for the failure of revetments. A protective apron or placing heavy riprap at the toe of the structure will minimize the risk of such failures. Figure 10 shows the typical section, with toe wall, used for the 1988 revetment project. A similar toe wall was installed for the 1977 and 1984 revetment projects. Table 6 presents specifications and amounts of the rock used in the toe wall for the revetment projects along LA-82. The amount of rock for the latter revetment project was small, because the limits of the 4-in.- and 8-in.-thick revetment projects overlapped (see Figure 4).

## SUMMARY AND CONCLUSIONS

Without failures, it is generally difficult to evaluate experimental coastal revetment systems. In lieu of actual revetment failures, several aspects of coastal revetment systems, as observed in the state of Louisiana's attempt to protect LA-82, will be discussed. These aspects include cost, ease of construction, performance, and recommendations.

### Cost

When dealing with limited funds and great lengths of coastal highway, costs become a critical factor. Table 4 presents the actual cost per square yard for the various revetment projects. Although the 8-in.-thick revetments are far more expensive than the earlier 4-in.-thick revetments, the cost of the 1988 8-in.-thick revetment project is far less than that of the original 8-in.-thick revetment project. The lower cost of the later project is most probably associated with the availability of revetments and the allowing of alternate types of revetments for the project. As shown in Table 4, the price of the Type F cast-in-place cellular block revetment is of great importance. This revetment cost 25 percent less than the precast blocks installed at the same time.

### Ease of Construction

The introduction of the cabled blocks allowed the installation of segments of revetment. The segments of revetments com-

TABLE 5 GEOTEXTILE PROPERTIES FOR THE REVETMENT PROJECTS ALONG LA-82

| PROPERTIES | FABRIC NO. 1 | FABRIC NO. 2 | FABRIC NO. 3 |
|---|---|---|---|
| Thickness, mils | 19 | 25 | 32 |
| Mass per Unit Area, $oz/yd^2$ | 7 | 5.3 | 6.0 |
| Open Area, percent | 5-6 | 20 | 20-30 |
| Apparent Opening, mm (US Sieve) | (70) | 0.42 | (40-50) |
| Permittivity, $Sec^{-1}$ | 0.40 | 1.99 | 0.90 |
| Puncture, lbs | 140 | 90 | 110 |
| Burst Strength, psi | 530 | 410 | 500 |
| Grab Tensile/ Elongation, lbs/percent | 390x280/ 28x25 | 300x215 | 340x240/ 32x25 |
| UV Resistance, % strength retained | 90 | 90 | 90 |

NOTE: Fabrics Number 1 and 2 were used in the 1969 test project. Fabric Number 2 was used with the four-inch revetment in 1970 and 1977. Fabric Number 3 was used with the eight-inch revetment in 1984 and 1988.

Properties listed in this table were obtained from Reference No. 12 and geotextiles which are similar to those installed under the various projects.

prised precast cellular blocks, which are cabled together into a mat. The mat is shipped to the project site and lifted into place with heavy construction equipment. This method of installation increases the ease of construction, but also makes repairs that require mat relayment uneconomical.

Heavy construction equipment is not needed with Type F cast-in-place cellular revetment. This fact makes this type of revetment more economical, as reflected in Table 4.

### Performance

The 4-in.-thick revetment installed in 1970 and 1977 did not have the weight necessary to resist uplifting forces. Although failure of this revetment system required the use of heavier blocks on the 1984 project, sections of the original 1970 projects were still functioning, and were simply covered over during the 1988 revetment project.

The cabled and noncabled blocks installed in 1984 have undergone several storms and have performed satisfactorily. The spaces in and between the blocks have been filled by accreted sand and shell fragments, making displacement of individual blocks unlikely. Although this accretion has consolidated the blocks into single-revetment systems, the accretion has not stopped the revetments from adjusting to movements in the sand under the blocks.

The revetment systems installed in 1984 and 1988 along LA-82 have undergone numerous serious storms without damage. The only damage to LA-82 had been associated with the scouring of the landside shoulder. Having successfully armored the gulf-side foreslope, the landside shoulder has now become the most vulnerable component of the protection system. Although the soil-cement foreslope treatment installed in 1977 successfully protected the landside of the embankment, the tides and waves created by hurricane Chantal destroyed portions of the shoulder, soil cement, embankment, and roadway.

TABLE 6 SPECIFICATION FOR ROCK TOE

Year installed: 1977
Project length: 18,514 feet
Quantity of rock: 36,863 tons
Rock size:

| Percent by Weight | Individual Stone Weight (pounds) |
|---|---|
| 10 | 1,000 |
| 20 | 500 |
| 30 | 300 |
| 25 | 150 |
| 15 | 70 |

Year installed: 1984
Project length: 9,200 feet
Quantity of rock: 8,733 tons
Rock size:

| Percent by Weight | Individual Stone Weight (pounds) |
|---|---|
| 40 | 1,250 - 2,500 |
| 25 | 500 - 1,250 |
| 20 | 100 - 500 |
| 15 | under 100 |

Year installed: 1988
Project length: 12,000 feet
Quantity of rock: 3,684 tons
Rock size: Same as 1984

**Recommendations**

Although the recent attempts to date to protect LA-82 have been successful, additional work by the Louisiana Department of Transportation and Development will be necessary to adequately protect the highway. During the research, numerous aspects of design were discovered that were not adequately addressed because of the lack of data at the time of design. The following are recommendations based on observations made by the authors:

1. Detailed wave and stage data should be acquired. Most failures are associated with lack of data.
2. The existing offshore profile should be surveyed. This task should also be repeated periodically to monitor the performance of the detached breakwaters as well as the revetments.
3. Sediment transport phenomena should be investigated.
4. Slope stability, influenced by hydrostatic pressure, should be studied.
5. Adequate protection for the landside foreslope should be provided to prevent its erosion at the time of overtopping.

## ACKNOWLEDGMENTS

The authors acknowledge the encouragement provided by Ara Arman, Steve Cumbaa, and Jack Manno, and the valuable comments received from Henry Barousse, Bill Jack, John Oglesby, Paul Griffin, Joseph Suhayda, and Dena Aubin. The authors also wish to express their thanks to Karen Jarreau for the graphics work presented.

## REFERENCES

1. H. Horikawa. *Coastal Engineering*. John Wiley, New York, 1978.
2. K. W. Pilarczyk, J. V. Overeem, and W. T. Bakker. Design of Beach Nourishment Scheme. *Proc., Coastal Engineering*, ASCE, New York, 1986.
3. *Artificial Structures and Shorelines*. H. J. Walker, ed., Kluwer Academic Publishers, Dordrecht, Netherlands, 1988, pp. 629–640.
4. L. D. Nakashima, J. Pope, J. Mossa, and J. L. Dean. Initial Response of A Segmented Breakwater System, Holly Beach, Louisiana. *Proc., Coastal Sediments, '87*, ASCE, New York, 1987, pp. 1399–1414.
5. *Survey of Holly Beach and Vicinity*. U.S. Army Corps of Engineers, New Orleans, La., 1971.
6. A. L. Cox. *Paving Block Study*. Research Report 58, FHWA, U.S. Department of Transportation, 1971.
7. R. L. Wiegel. Trends in Coastal Erosion Management. *Shore and Beach*, Vol. 55, No. 1, Jan. 1987, pp. 3–11.
8. *Shore Protection Manual*. U.S. Army Corps of Engineers, Volumes I and II, Washington, D.C., 1984.
9. *Design and Construction of Mounds for Breakwaters and Coastal Protection*. Per Bruun, ed., Elsevier Science Publishers, 1985.
10. L. Dement. Two New Methods of Erosion Protection for Louisiana. *Shore and Beach*, Vol. 45, No. 1, 1977, pp. 31–38.
11. *Geotextile Engineering Manual*. National Highway Institute, Washington, D.C., 1985.
12. *Geotechnical Fabrics Report*. Specifier's Guide, Vol. 6, No. 7, Saint Paul, Minn., 1989.

*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# April 1989 Hatchie River US-51 Bridge Failure

PHILIP L. THOMPSON

The FHWA assisted in the National Transportation Safety Board (NTSB) determination of the cause of the collapse of the spans of the northbound US-51 bridge over the Hatchie River on April 1, 1989. The collapse resulted in five vehicles going into the river and eight people being killed. The bridge site, field observations, stream stability, analysis of aerial photographs, model studies, and foundation analysis are discussed.

Spans of the northbound US-51 bridge over the Hatchie River, referred to as the "old bridge," collapsed on April 1, 1989. Five vehicles went into the river and eight people were killed as a result of the collapse. The National Transportation Safety Board (NTSB) initiated an investigation into the cause of the collapse and requested that the FHWA be a party to that investigation. FHWA actions to assist the NTSB determination of the cause of the collapse of the bridge are described.

## BRIDGE SITE

The bridge site is located about 10 mi north of Covington, Tennessee, where US-51 crosses the Hatchie River (see Figure 1). At this crossing, the northbound lanes are carried over the floodplain of the Hatchie on the old bridge, which was built in 1936, and the southbound lanes are carried on the new bridge, which was designed in 1974 and built thereafter. The old bridge was approximately 4,200 ft long and consisted of 137 simple spans of 28 ft 6 in. over the floodplain and six longer spans over the main channel (see Figure 2). The main channel spans were supported on foundations significantly deeper than the floodplain spans. The bottom of footing elevation was approximately at elevation 223 for Piers 1 to 7 and at elevation 237.9 for pier 70 (Figure 3). Each bent had two columns. Each column had independent footings with five untreated timber piles in each footing that were 20 ft long. This bridge is approximately 1,000 ft long. It has 12 bents and 13 spans. The 11 interior spans are 76 ft 9 in. and the two end spans are 77 ft 7 in. The new bridge, similar to the old bridge, has foundations in the main channel that are much deeper than those located on the floodplain. Bents 6 through 8 have bottom of footing elevations at about 226.2 ft, while the remainder are at elevation 240 ft. Each bent has three columns. Each column has an independent footing that is 8 ft 6 in. square and six prestressed concrete piles that are 30 ft long.

FHWA, U.S. Department of Transportation, 400 7th St., S.W., Rm. 3113, Washington, D.C. 20590.

## FIELD OBSERVATIONS

The author and J. Sterling Jones of the FHWA were on the site during the week beginning April 3, 1989. They participated in all phases of the investigation with Joseph Osterman, NTSB investigator-in-charge, and Lawrence E. Jackson, NTSB highway group chairman. The major activities of the week were monitoring recovery, data gathering, interviewing the bridge inspection crew, site reconnaissance by boat, and underwater inspection with the FHWA Demonstration Project 80 (DP 80) team and boat.

### Monitoring Recovery

Most of the week was spent with the recovery operation shown in Figure 4. The recovery of vehicles was extremely difficult

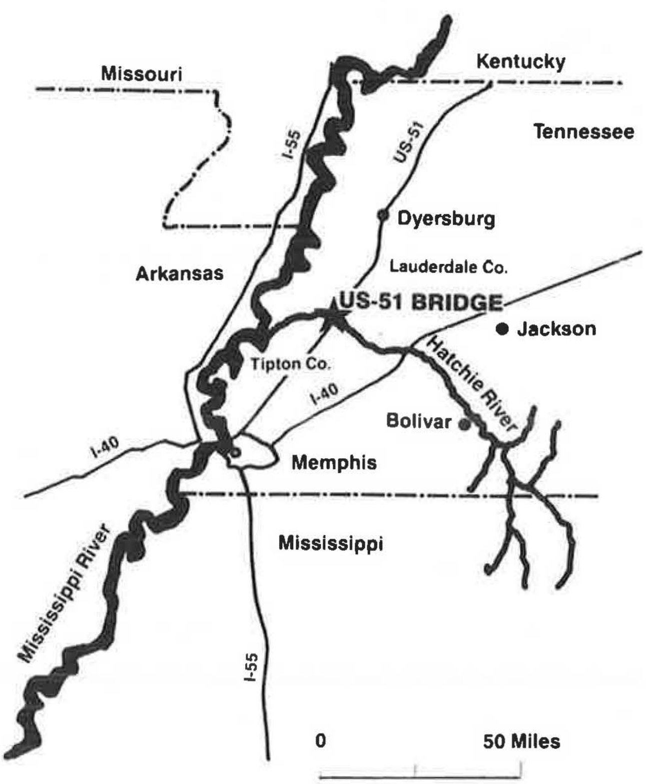



FIGURE 1 Location map (NTSB, 1990).

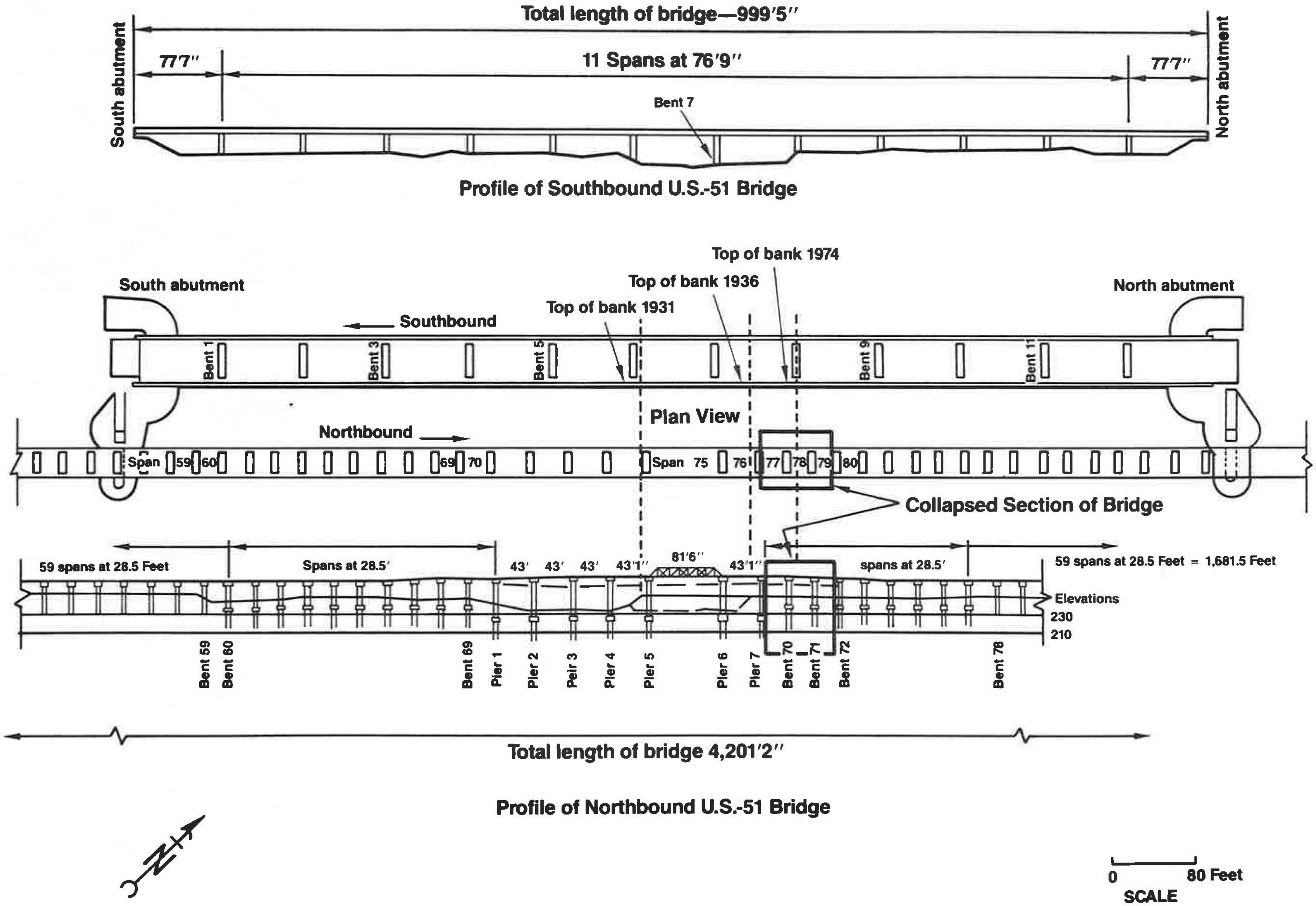



FIGURE 2 Layout of bridge (1931 and 1974) (NTSB 1990).

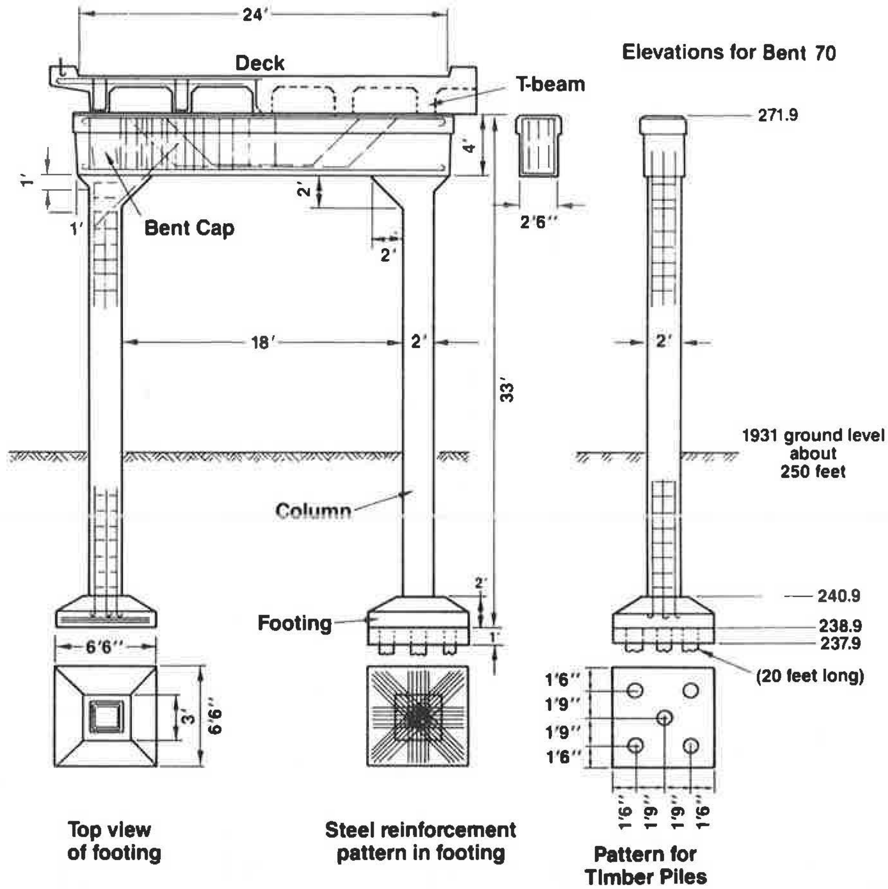



FIGURE 3 Concrete Bents 1 through 135 (NTSB 1990).

because of the sequence of the collapse. Altogether, three spans (77 through 79) similar to Span 80, which is shown in Figure 4, collapsed into the river along with Bents 70 and 71. Apparently, Bent 70 collapsed first with Spans 77 and 78. Three vehicles fell through the opening and landed on the north bank under Span 79. A truck (tractor-trailer) was apparently next to fall through the opening. The truck struck Bent 71 and apparently caused it to collapse across the tractor. Span 79 fell on top of these vehicles. The last vehicle to fall in was recovered from the channel near the original location of Bent 70.

Figure 5 shows the collapsed section of the bridge as it appeared from the direction that traffic approached it. Skid marks were apparently left by the truck before it went through the opening. State troopers checked levels on the span that was standing on the south side and found no measurable slope,

FIGURE 4 Recovery operation.

FIGURE 5 Collapsed spans viewed from direction of traffic.

**FIGURE 6 Span 79 debris removed.**

**FIGURE 7 Location of truck wreckage.**

which suggests that Pier 7 at the north end of that span did not settle. Pier 7 is supported by the deeper foundation of main-channel type.

In order to recover the vehicles, Span 79 had to be broken apart. Small sections of the deck and supporting integral girder were freed from the span by jack hammering a slot in the concrete and cutting the reinforcing bars. These pieces were deposited either on the north bank between the old and new bridges or east of the old bridge. The three vehicles (green Pontiac, grey Toyota, and grey Ford) were recovered in this manner. To get to the truck, the remainder of the west facia girder was cut away and deposited with the other debris. The remainder of the span was just east of the old bridge (Figure 6). Recovery operations were then suspended on Thursday until divers could document the location of the west column of Bent 70, which was lying across the truck. Figure 7 shows the pier cap of Bent 71. The west column extended from the pier cap to the left across the truck. The front left truck tire is visible in Figure 7 and the front right hub is located to the right with the front axle in between. The back wheels of the truck were on Span 78 near where the diver is standing. Friday morning, divers from Collins Engineers, Inc., recorded the location of the column (see underwater survey). The column was then lifted off the truck, and the truck was hoisted onto a trailer. Recovery was complete by Friday afternoon.

### Data Gathering

The major sources of background information were the Tennessee Department of Transportation (TDOT), the U.S. Geological Survey (USGS), and the Corps of Engineers (COE). An additional source of information was Herb Murphy, a diver. The TDOT provided the most recent inspection report (1987) both of the old and new bridges and full-sized bridge plans. These documents were the primary reference documents used during the week. Later, all existing inspection reports (of 1971, 1979, and 1985) were provided along with the hydraulic study for the new bridge.

Harry Doyle, Jr., Chief of the Memphis subdistrict of USGS, met with the team on site and negotiated a statement of work for a stream stability study of the reach of the Hatchie River in the vicinity of the bridges. Findings of this study are discussed later. It was noted that the flow was out of the main channel for the entire week. It was later determined that this flood was not a major event and had a recurrence interval of less than 2 years. Overbank flow is typical of this reach of the Hatchie River, and these types of flows are typical much of the time between November and May of each year. Overbank flow at this site is expected to be sustained for months and does not recede in a few days as with typical riverine flooding. A USGS survey crew was observed on site during the week taking velocity and cross section measurements.

The COE, Memphis office, operates a recording gauge mounted on Pier 6 of the old bridge. The COE provided printouts of daily stages and discharges from 1939 through April 3, 1989. In addition, the COE provided all existing aerial photography of the site. The flow data and aerial photography are discussed later.

Herb Murphy, a local diver, assisted in recovering bodies, recovering vehicles, and doing underwater reconnaissance of the collapsed spans. In addition, he was hired by TDOT to perform underwater inspection of the new bridge. He helped the team identify relative locations of vehicles and major bridge members. He briefed the divers from Collins Engineers, Inc., and assisted in the underwater survey of bridge members.

### Bridge Inspection

The team interviewed the TDOT bridge inspection crew that prepared the 1987 bridge inspection report. The crew chief described the step-by-step procedures used during the inspection. They noted the soundings taken around Bent 70 (Figure 8) and described how the scour depths that are shown on the elevation view of Bent 70 (Figure 9) were determined. Apparently, a range pole was used to locate the top of footing about 1 ft underwater and stream bottom about 7 ft underwater. The footing was determined to be 5 ft thick, probably by probing. The scour depth was then calculated to be 1 ft. Because the plans show the footing to be 3 ft thick, the actual scour depth below the footing was 3 ft at the time of inspection.

The inspectors confirmed that drift had accumulated on Bents 70 and 71. The reproductions of the photographs in the

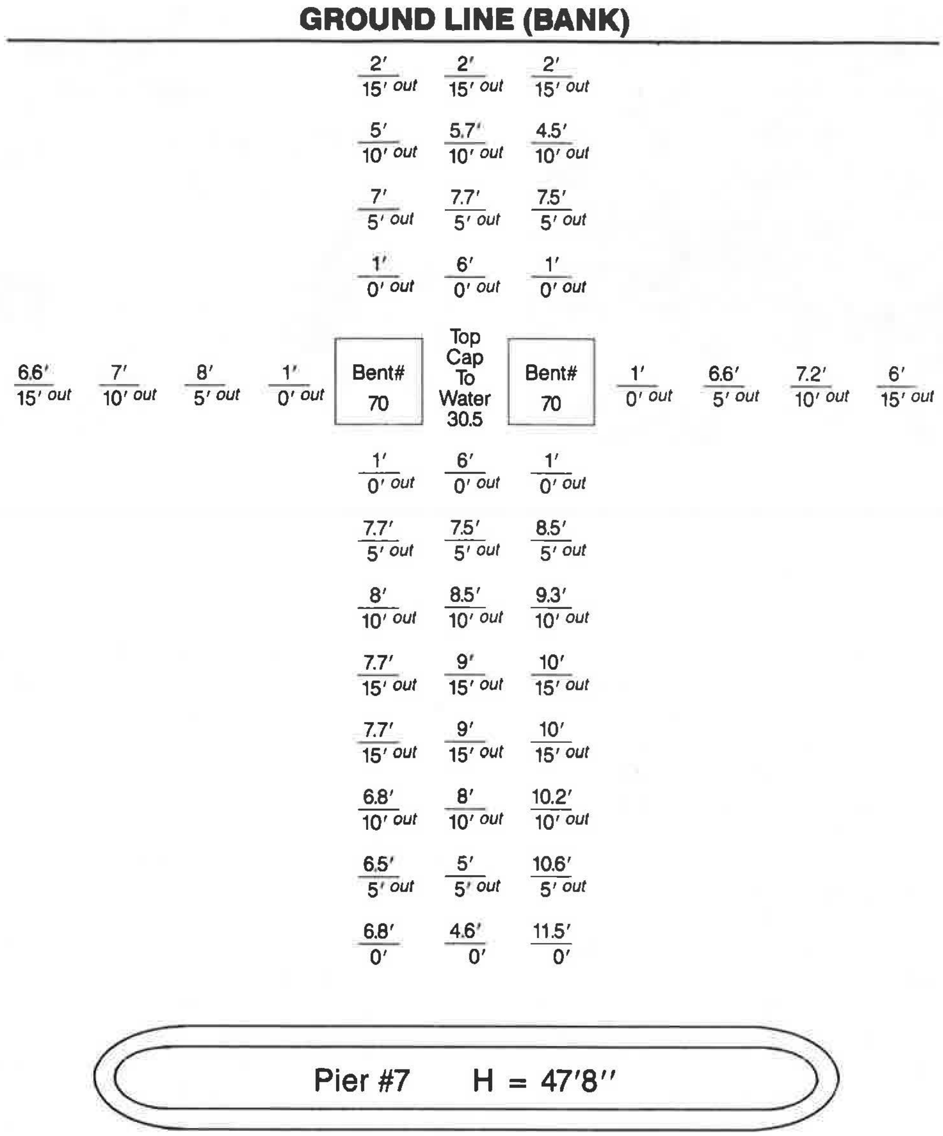



**FIGURE 8 Soundings at Bent 70 (NTSB 1990).**

inspection report were not clear enough to see the drift. The inspectors also confirmed the following two maintenance recommendations in the 1987 report:

1. Clear drift, and
2. Protect Piers 5 through 7 and 70 from scour.

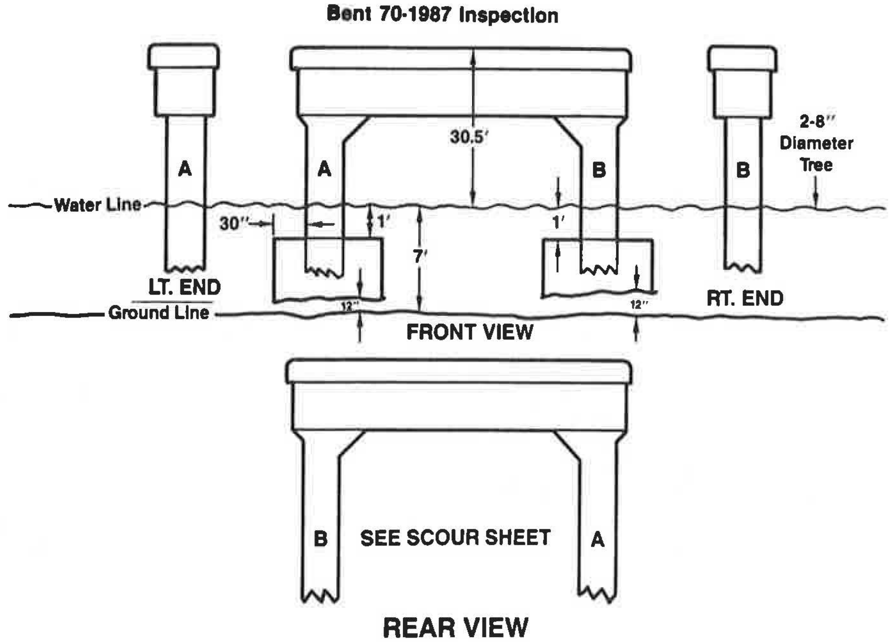
Similar recommendations are also contained in the 1985 inspection report that was obtained subsequently. All existing inspection reports before 1985 were also reviewed, but no others contained similar recommendations.

### Site Reconnaissance

Messrs. Thompson and Jones, with the assistance of the TDOT inspection crew, used the TDOT boat to assess the Hatchie River main channel using a black-and-white fathometer and a sounding weight. Cross sections were taken upstream, downstream, and at the bridges. The channel was determined to have steep banks and have water depths of 15 to 18 ft on the inside of bends and 20 to 25 ft on the outside of bends. The depth of flow in the trees at the banks varied from 6 to 8 ft. Soundings at Pier 7 indicated flow depths of about 22 ft. This would put the bottom about 39 ft below the pier cap. The total height of Pier 7 is 46 ft 8 in. Therefore, Pier 7 was assessed as safe. This assessment was later confirmed by divers from Collins Engineers, Inc. The location of the north bank in the vicinity of the bridges was established. The bank was between Piers 8 and 9 of the new bridge, about midspan. This location was a concern because Pier 9 is a floodplain pier with footings above the bottom of the stream.

### Underwater Investigation

The FHWA DP 80 crew and boat were used on Friday and Saturday to survey the Hatchie River and the collapsed mem-




| ELEMENT | | RATING | COMMENTS |
|---|---|---|---|
| BEARING | | G F P C | N/V |
| CAP | | G(F)P C | Water Abrasion |
| COLUMN | A | G(F)P C | Water Abrasion |
| | B | G(F)P C | Water Abrasion |
| SCOUR | | G F(P)C | Washed Under Footing |

**FIGURE 9 Scour depths at Bent 70 (NTSB 1990).**

bers of the old bridge. The crew included divers from Collins Engineers, Inc., who located some of the collapsed bridge members and inspected Pier 7. The DP 80 boat was used both to make a black-and-white fathometer and a subbottom profiler strip chart recording of the bottom.

Collins Engineers' findings are documented (*1*). Two figures from that report are reproduced for this summary: Figure 10 is an elevation view of the old bridge in the vicinity of the collapse; Figure 11 is a plan view in the same location. These two figures show the position of the members that could be located.

Figures 9 and 10 have not been updated as the result of a subsequent investigation that was accomplished when the water was only 12 ft deep. This investigation determined that the footings of Bent 70 were located on the stream bottom between Bent 70 and Pier 7. The footings were broken free both from the column and piles and were resting flat on the bottom. A pile stub was accessible on the east footing, northwest corner. This pile was measured to be 11 in. in diameter (standards called for 12 in.). The only other pile found was in the vicinity of the original location of Bent 70, east side. However, this pile was split into three parts and could not be measured. The pile material appeared to be sound. Samples were taken for analysis by the Forest Service, Forest Products Laboratory.

## STREAM STABILITY

On the basis of the depth of water where recovery divers were working, and later confirmed by sounding made by FHWA's bridge inspection demonstration project team, the north bank of the main channel was somewhere between Bent 71, which collapsed, and Bent 72, which remained standing. Figure 12, a cross section of the floodplain and bridge foundations, shows the probable progression of the north bank with time. A short reach of the channel was straightened in 1932 when the old bridge was built to improve alignment of flow through the bridge. The main channel was designed to be between Piers 5 and 6. These piers have the deeper main channel footings that are supported by concrete piles. Pier 7, which was designed to be close to the bank line, has a deep foundation. According to the plans, it is supported by untreated timber piles. The two bents that collapsed had shallower footings and pile cutoff elevations approximately 16 ft higher than the main chan-

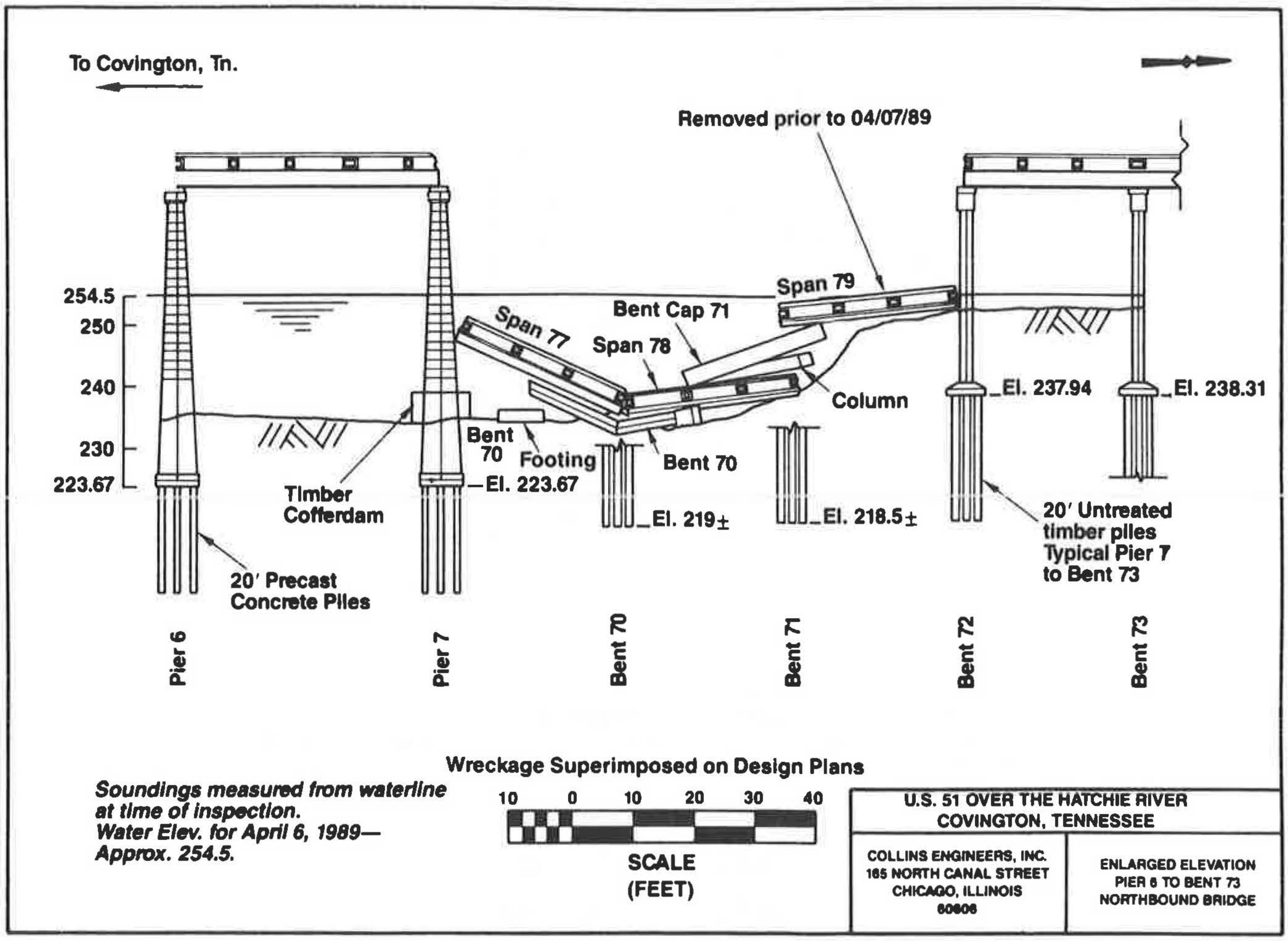



**FIGURE 10 Elevation view of collapsed spans.**

nel footings. As the channel bank migrated past Bent 70, the footing and the piles were exposed to the main channel flow.

Having piles exposed to the flow is not necessarily a critical situation, as there are bridges that are designed to be stable with exposed piles. This particular bridge was not designed that way. For this bridge, the structural integrity of the piers as well as the supporting capability of the friction piles were in jeopardy when the bank line migrated past Bent 70.

The USGS study of stream stability of the Hatchie River in the vicinity of the old and new bridges (*2*) indicates that the narrowing of the floodplain by the new bridge in 1974 caused contraction scour (in the form of widening) to occur in the channel in the vicinity of both bridges. The widening is demonstrated by plotting the channel width versus time (Figure 13).

The USGS study indicates that the discharges during February and March were not abnormal. The discharge at the time of collapse was approximately 8,600 ft$^3$/sec, and this was not the peak of this flood event. The peak occurred 2 days later, on April 3, and was about 11,900 ft$^3$/sec. This peak has a recurrence interval that is less than 2 years. In fact, it is likely to occur every year. The water surface elevation at the time of collapse was approximately 254.3 ft, and the peak was about 0.7 ft higher according to COE gauge records.

In order to determine if the channel widening or migration was either local to the bridge or occurring throughout this reach of the Hatchie river, a study was made of available aerial photography.

## ANALYSIS OF AERIAL PHOTOGRAPHS

The Memphis district of the COE furnished aerial photos that covered the period from 1948 to 1984. Because many of the streams in Western Tennessee are unstable and migrate with time, significant changes in the river patterns were expected, but this instability was not the case. The Hatchie River is a stable stream. Many of the channel patterns that were evident in 1948 were still clearly visible in the 1984 photographs. Figures 14–19 are the aerial photos for 1948, 1973, 1974, 1976, 1979, and 1984, respectively. The US-51 bridge was slightly off of the 1948 photo.

The conclusion that the Hatchie River is stable was made after comparing the photographs. First, a reach of channel was selected for comparison. Because the railroad line was a common fixed feature on all the photos, it was used as a reference. The photographs were reduced and the river reach in the vicinity of the railroad was compared using the pho-

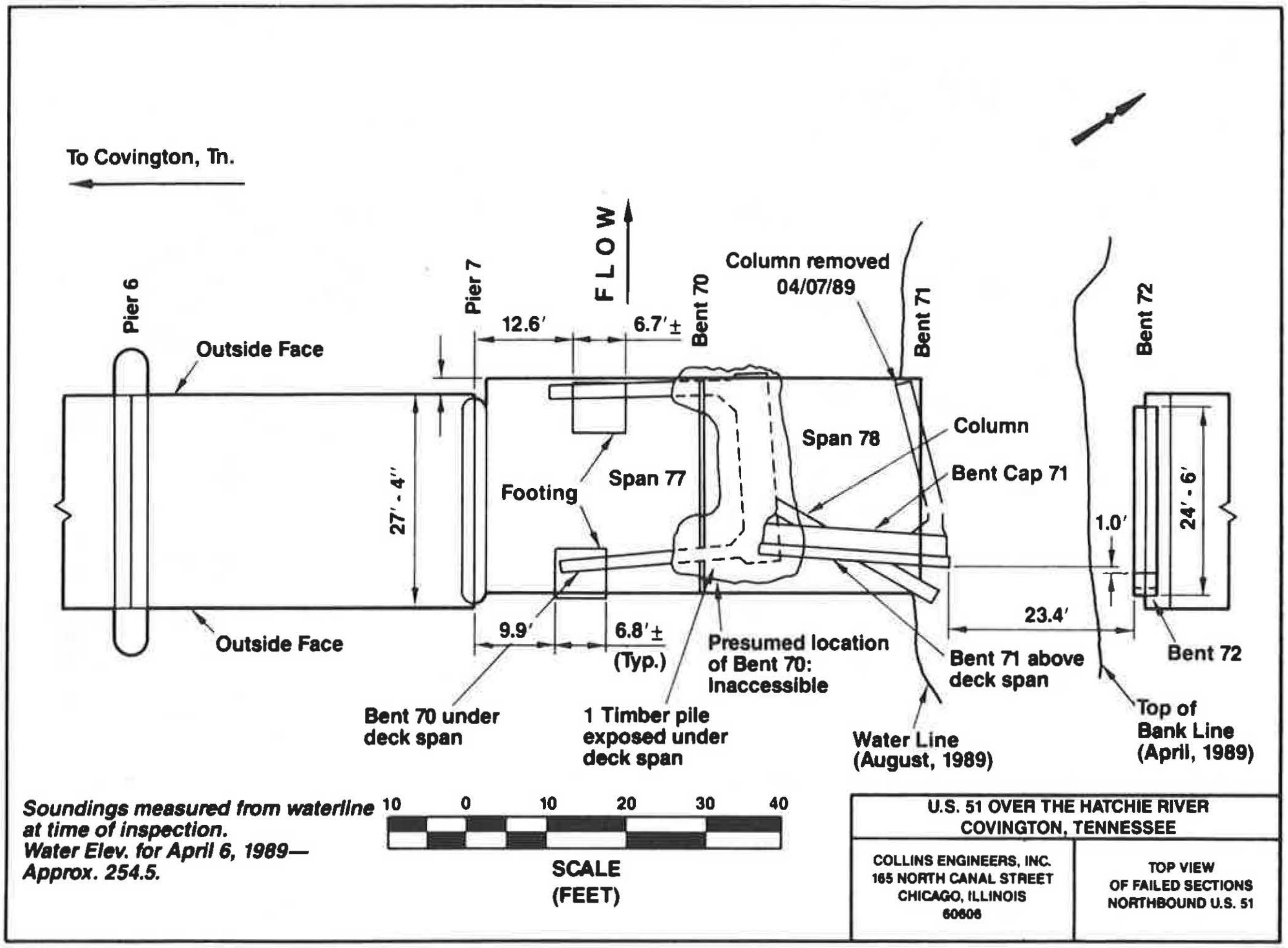



**FIGURE 11** Plan view of collapsed spans.

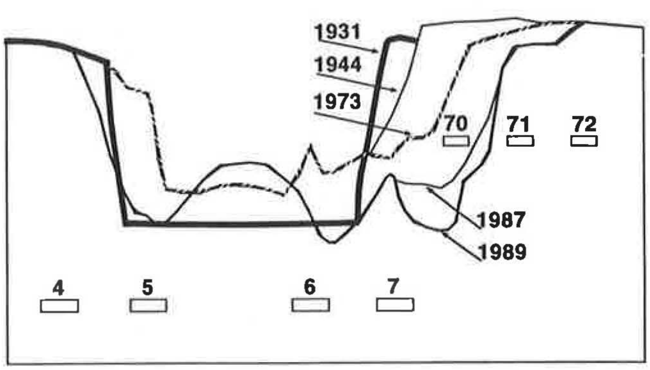



**FIGURE 12** Probable channel migration.

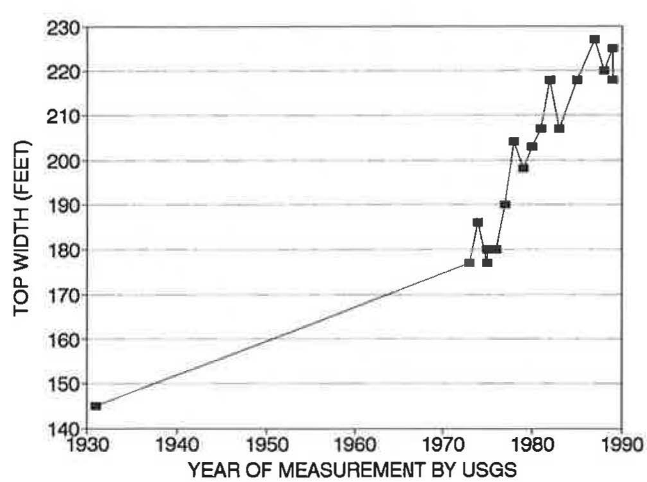



**FIGURE 13** Channel widening.

**FIGURE 14** Aerial photograph, Oct. 13, 1948 (channel $Q$ = 642 ft$^3$/sec, floodplain $Q$ = 0).

**FIGURE 15** Aerial photograph, April 2, 1973 (channel $Q$ = 9,218 ft$^3$/sec, floodplain $Q$ = 18,420 ft$^3$/sec).

**FIGURE 16** Aerial photograph, Oct. 17, 1974 (channel $Q$ = 942 ft$^3$/sec, floodplain $Q$ = 0).

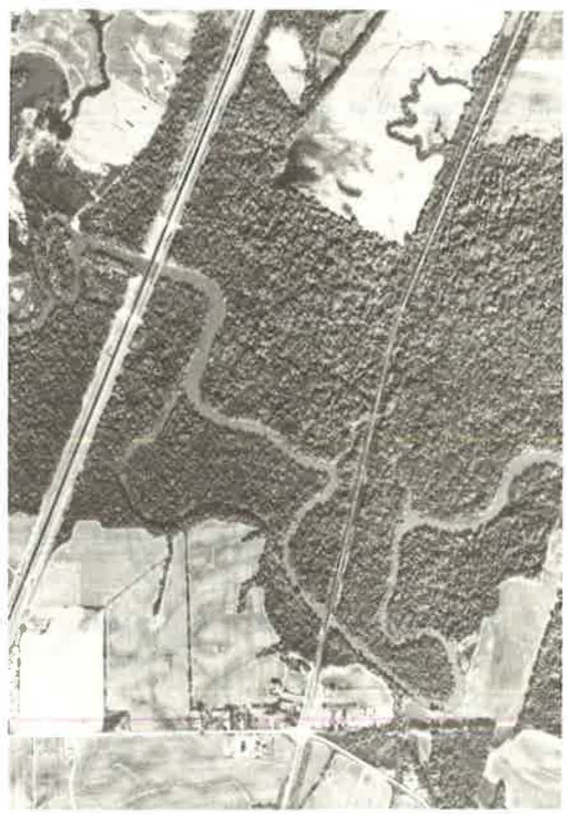


**FIGURE 17** Aerial photograph, Nov. 4, 1976 (channel $Q$ = 2,060 ft$^3$/sec, floodplain $Q$ = 0).

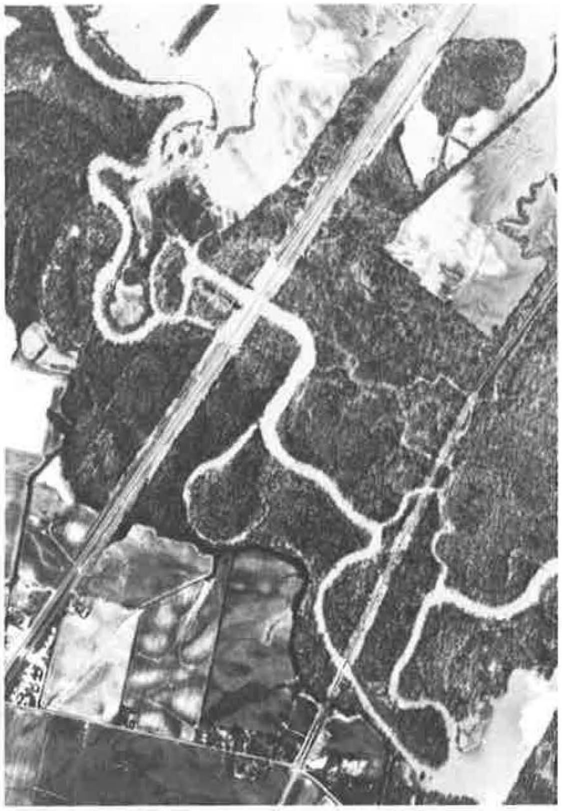


**FIGURE 18 Aerial photograph, March 6, 1979 (channel $Q$ = 7,001 ft$^3$/sec, floodplain $Q$ = 2,653 ft$^3$/sec).**

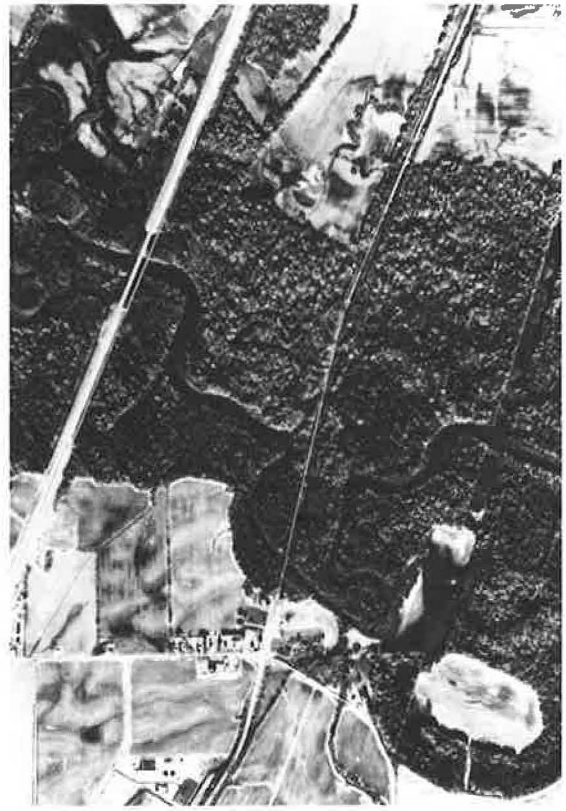


**FIGURE 19 Aerial photograph, Dec. 3, 1984 (channel $Q$ = 2,968 ft$^3$/sec, floodplain $Q$ = 1,289 ft$^3$/sec).**

tographs of 1948, 1973, 1979, and 1985 (see Figure 20). The 1985 blowup of a small-scale aerial photograph was substituted for 1984, because the channel was clearer.

After reviewing Figure 24, it is clear that the river patterns on each side of the railroad line, which shows up on all the photos, changed little during that 37-year period.

In summary, the aerial photographs indicate that the Hatchie River was an unusually stable stream for western Tennessee. The channel migration that did occur was confined to the vicinity of the bridge (see Section 4) and is not readily discernible from a systematic study of the aerial photographs.

## MODEL STUDIES

The FHWA was asked by NTSB to conduct a hydraulic model study of Bent 70. A study (*3*) was carried out at the FHWA Turner Fairbank Highway Research Center, Hydraulic Laboratory.

The study determined that the depth of scour for Bent 70 with the piles exposed (Figure 21) is 3.3 ft at the center pile of the upstream footing (Figure 22). The scour is about 40 percent less at the downstream footing. The model was verified by determining the scour for the square column and comparing this value (4.3 ft) with the value (5.1 ft) obtained from the pier scour equation in FHWA's Technical Advisory T5140.20, *Scour at Bridges*. The equation used was the Colorado State University (CSU) equation for scour at solid shaft piers.

Debris was added to the piles to simulate accumulated drift. The debris was not dense and did not appreciably increase the scour depth. In practice, if the debris was a dense clump or perhaps a solid tree trunk, the scour hole could have been much deeper.

## FOUNDATION ANALYSIS

The foundation analysis was documented by Dimaggio (*4*). The analysis was accomplished for Bent 70 assuming that the pile material was Douglas fir, that the piles used had a 12-ft butt diameter and an 8-in. tip diameter, that the bottom was at Elevation 231, and that the piles were 20 ft in length.

The analysis indicates that with these conditions and with a little more than 4 ft of local scour, the piles do not have sufficient axial load capacity and could settle. Therefore, this is the likely mode of failure of Bent 70.

If the pile material is sound, neither lateral pile capacity nor pile buckling is a likely mode of failure. However, depending on the actual unsupported pile length, the condition of the pile material, the size of pile, and the type of pile material, these conclusions may change. For example, a pile buckling failure could occur if the pile diameter is reduced 25 percent from 12 to 9 in.

Bent 9 of the new structure was also analyzed because the north bank is approaching this bent. Bent 9 was determined to be close to unstable for the conditions outlined. Therefore, the floodplain bents of this structure should have countermeasures to protect the foundations from scour.

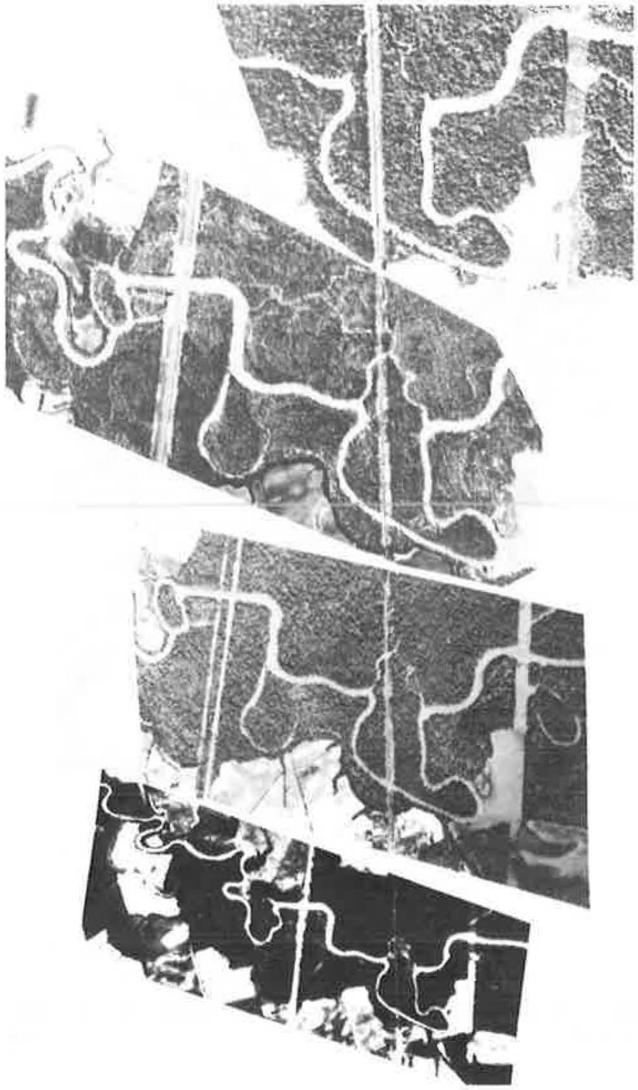


FIGURE 20 Aerial photography comparison.

FIGURE 21 Model of Bent 70.

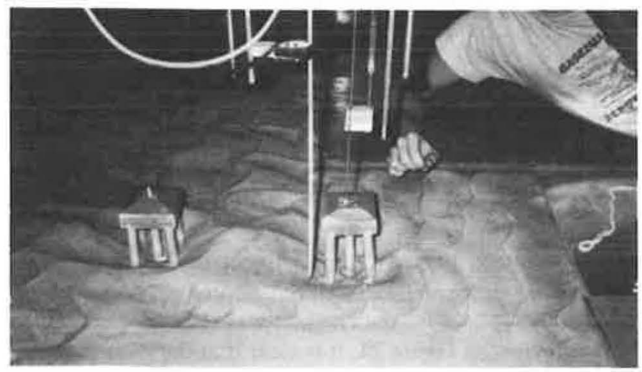


FIGURE 22 Model with scour.

## FINDINGS AND RECOMMENDATIONS

The following observations are preliminary and may change with subsequent findings from the recovery of the bridge wreckage that is scheduled for December or from the NTSB hearings that are scheduled for November 28 and 29.

Finding. The floodplain pier foundations were located at a higher elevation than the channel pier foundations and the channel banks were unconstrained. Either of those conditions by itself poses no particular risk if the stream is stable. However, in combination with an unstable stream reach, the foundation in this case was at risk.

Recommendation. Channel pier foundations and adjacent floodplain pier foundations both should be located below the channel thalweg, unless the channel banks are armored or the channel has exhibited long-term stability and can be assessed to remain stable.

Finding. Sounding data were taken for all piers in the channel during regular inspections. This information was not transferred to a cross section plot which included pier location and foundation elevations.

Recommendation. A cross section of the channel should be plotted after each inspection. The plot should include appropriate substructure information. The cross section should be compared to those taken in previous years so that stream changes can be identified. If movement has occurred, it should be assessed by a hydraulic engineer.

Finding. The most recent inspection report identified that the streambank had moved and that scour had occurred below the footing. However, neither the project plans nor other inspection documents identified this as an unsafe condition for this structure.

Recommendation. The critical scour elevation should be identified for unprotected piers and bank migration limits identified for streams without revetment.

## REFERENCES

1. Collins Engineers, Inc. *Underwater Investigation of the U.S. Route 51 Northbound Bridge over the Hatchie River at Covington, Tennessee.*
2. USGS. *Channel Evolution at the U.S. Highway 51 Crossing of the Hatchie River, Lauderdale and Tipton Counties, West Tennessee,* 1989.
3. J. S. Jones. *Model Study of Hatchie River U.S. 51 Bridge, Bent 70.* FHWA, U.S. Department of Transportation, Sept. 1989.
4. J. Dimaggio. *April 1989 Hatchie River U.S. 51 Bridge Failure.* FHWA, U.S. Department of Transportation, 1989.

*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# Pennsylvania State Urban Hydrology Model as a Tool for Highway Drainage Design

THOMAS A. SEYBERT AND DAVID F. KIBLER

The capabilities of the Pennsylvania State Urban Hydrology Model are presented. The model contains 17 subprograms that perform various hydrologic and hydraulic operations, which include curve number weighting, travel time, swale design, hydrograph combining, graphics plotting, rainfall calculations, Soil Conservation Service (SCS) unit hydrograph, SCS tabular hydrograph, modified rational hydrograph, Muskingum channel routing, modified Puls routing, and multiple-stage outlet analysis. The model operates on an IBM personal computer or compatible and is menu driven. An example is presented to illustrate the use of the model.

The Pennsylvania State Urban Hydrology Model (PSUHM) is a menu-driven interactive model written in the BASIC programming language specifically for IBM-compatible MS–DOS personal computers. The model contains 17 programs that perform various hydrologic tasks. The tasks are organized into four types of hydrologic operations; namely, utilities, rainfall, runoff, and routing, as indicated by the PSUHM main menu screen shown in Figure 1. The programs are connected through a submenu selection program and a data file management system. Several of the programs, which are modules for the PSUHM, are capable of saving the computational results of the selected task in an MS–DOS file. These data can be accessed by other modules later through the data file management system. Data that can be saved as a file include hyetographs, hydrographs, subarea characteristics, elevation-storage charts, elevation-outflow charts, and detention outlet geometry.

The first version of the PSUHM was issued in October 1984 in a Pennsylvania State University continuing education short course on computational methods in stormwater management. This original software has been modified substantially through annual revisions. The present hydrologic and hydraulic capabilities in PSUHM include design storm and hyetograph calculations, curve number weighting, rational and modified rational methods, Soil Conservation Service (SCS) curvilinear unit hydrograph, SCS TR–55 tabular hydrograph, Muskingum channel routing, modified Puls reservoir routing, hydrograph combining, and plotting and sizing of outlets in a multistage detention structure.

The multistage outlet design and routing model in the PSUHM is especially useful in light of recent ordinances requiring control of multiple points on the flood frequency curve. The outlet routine is capable of developing a complex rating curve representing up to 10 different outlets or stages. Output takes the form of elevation-storage-discharge tables that can be passed to the reservoir routing module for full analysis of a proposed detention facility.

T. A. Seybert, Department of Engineering, Beaver Campus, Pennsylvania State University, Brodhead Road, Monaca, Pa. 15061. D. F. Kibler, Department of Civil Engineering, Patton Hall, Virginia Polytechnic Institute and State University, Blacksburg, Va. 24061.

## DESCRIPTION OF PSUHM MODULES

In order to access any of the 17 modules in PSUHM, the user must select the group number on the main menu screen that contains the module of interest. Each module in PSUHM will be briefly described here according to the four basic groups of the main menu. More detailed information on each module is provided in the PSUHM *User's Manual* (*1*).

### Utility Modules (Group 1)

A utility module is defined here as any module that does not perform rainfall, runoff, or routing calculations. It is the miscellaneous category of PSUHM. There are five modules in this group.

Module 1A. *Curve Number (CN) Weighting* is an area-based weighting scheme that computes an average CN value for a multiple–land use subarea. The module allows up to 20 land uses per subarea and an unlimited number of subareas. CN values are used to estimate runoff from rainfall and are devel-

```
****************************************
*  PENN STATE URBAN HYDROLOGY MODEL    *
****************************************

            ** MAIN MENU **

          HIT        TO RUN

          <1>       UTILITIES
          <2>       RAINFALL
          <3>       RUNOFF
          <4>       ROUTING

          <I>       information
          <D>       shell to DOS
          <Q>       to QUIT
```

**FIGURE 1 PSUHM main menu screen.**

oped by the SCS as described in the SCS Technical Release 55 (TR–55) (*2*).

Module 1B. *Travel Time Calculation* will solve various travel time equations that are listed in the module's travel time menu. This module features the SCS segmental approach to travel time calculation from TR–55 (*2*). Other equations solved in this module are the average velocity charts in the 1975 TR–55 (*3*), the Federal Aviation Administration (FAA) overland flow equation, the kinematic wave equation, Manning's equation for pipe or channel flow, and the SCS basin lag equation. These single equations are summarized in the American Geophysical Union (AGU) Water Resources Monograph No. 7 (*4*).

Module 1C. *Hydrograph Combination* will add two hydrographs together, as in the case of a confluence point in a stream network. The new hydrograph can be saved in a data file.

Module 1D. *Data File Plotting* uses screen graphics to display the contents of one or two data files of the same type for comparative analysis. Hyetographs, hydrographs, elevation-storage, and elevation-outflow files may be plotted on the display. The screen display can be sent to the printer.

Module 1E. *Swale Design* uses a Newton-Raphson scheme to solve Manning's equation for a trapezoidal channel. The user can specify depth and solve for flow or specify flow and solve for depth. A report of several hydraulic parameters of the swale is provided, including a recommended rip-rap for bed stability.

**Rainfall Modules (Group 2)**

Five rainfall modules are available in PSUHM. Every module creates a hyetograph that can be stored and used in conjunction with the Unit Hydrograph Module 3A.

Module 2A. *Composite Storm by U.S. Weather Bureau IDF Curves* will create a synthetic hyetograph for a specified return period, on the basis of the U.S. Weather Bureau's standard intensity-duration-frequency (IDF) curves as developed by Yarnell (*5*). The storm is central peaking, limited to a maximum length of 120 min and minimum step of 5 min.

Module 2B. *SCS Type II Scaled Rainfall Distribution* will create a synthetic hyetograph according to the SCS Type II rainfall distribution. This distribution is defined in TR–55 (*2*). The duration of the storm can be any length from 0 to 24 hr and the step can be any practical value, in hours. Design storms shorter than the Type II standard of 24 hr are scaled to include the most intense portion of the 24-hr distribution.

Module 2C. *SCS Type III Scaled Rainfall Distribution* works exactly the same as the Type II Module 2B, except it uses the Type III rainfall distribution also described in TR–55 (*2*).

Module 2D. *Composite Storm by Pennsylvania DOT–IDF Curves* works practically identical to Module 2A except that it uses rainfall curves developed specifically for Pennsylvania as given by Aron (*6*). Use is limited to Pennsylvania and neighboring areas.

Module 2E. *Manual Entry of a Hyetograph* will allow the user to enter a design storm or observed storm of any duration and any time step. The storm can contain up to 101 data points.

**Runoff Modules (Group 3)**

The runoff modules contain methods that create complete runoff hydrographs for a given watershed or subarea. The TR–55 methods in Module 3B were designed to create partial hydrographs; however, for most small watersheds the hydrographs produced will be sufficiently complete.

Module 3A. *Subarea Hydrographs by SCS Unit Hydrograph* will create a hydrograph based on the SCS curvilinear unit hydrograph discussed in the SCS *National Engineering Handbook*, Section 4 (*7*). A design storm created through one of the rainfall modules in Group 2 acts as the input to this subprogram. Along with the runoff hydrograph, periods of rainfall excess and the unit hydrograph are presented. The method is general and no limit is placed on watershed size.

Module 3B. *SCS TR–55 Tabular Hydrograph* will create a hydrograph using the 1986 tabular method found in Chapter 5 of TR–55 (*2*). The method is based on the SCS TR–20 runoff program and is an approximate method for developing a hydrograph for a watershed composed of several subareas. In addition to a complete hydrograph, this module generates a summary screen that shows runoff contributions of each subarea to the peak flow in the composite hydrograph. The method is limited to areas with times of concentration less than 2 hr, but greater than 0.1 hr.

Module 3C. *Hydrograph by Modified Rational Method* will create a simple yet useful hydrograph for small areas (0 to 20 acres) with uniform land use. The method uses the Pennsylvania Department of Transportation IDF curves (*6*) to create a skewed hyetograph with time base equal to 11 times the subarea time of concentration. The program then utilizes the rational formula ($Q = ciA$) by multiplying each ordinate of the hyetograph by $cA$ to create a hydrograph. The program also has an option that allows the user to estimate the required detention pond size for a given design release rate.

Module 3D. *Manual Entry of Hydrograph* will allow the entry of an observed or created hydrograph (up to 101 points) to be used with other PSUHM modules such as Module 4B Modified Puls Routing.

**Routing Modules (Group 4)**

The routing modules will pass a given hydrograph through a detention structure or channel reach and examine the effects on the hydrograph. PSUHM contains three routing modules.

Module 4A. *Muskingum Channel Routing* is a hydrologic procedure that routes a hydrograph through a channel reach. It is approximate and is based on the principle of mass continuity and a storage flow relation. The method is described in most hydrology texts, such as Viessman et al. (*8*). The module checks to ensure that the routing procedure is numerically stable and that the proper number of subreaches is used.

Module 4B. *Modified Puls Routing* will route a hydrograph through a reservoir using the storage indication method also described in Viessman et al. (*8*). This method is based on mass conservation principles and the hydraulics of the reservoir outlet structure. The user must develop and enter two curves: a storage-elevation curve from basin contour data and a discharge-elevation curve from the hydraulics of the outlet configuration. The storage-elevation and discharge-elevation curves can be stored in a file and used again for other runoff situations. The storage-elevation and discharge-elevation curves can be developed in Module 4C.

Module 4C. *Multiple Stage Routing Model (MSRM)* will route a hydrograph through a multiple outlet detention facility. Written by Chamberlain (*9*), the model contains a subroutine that will compute the hydraulic performance curve for an outlet structure with up to 10 openings or stages. This capability is useful when trying to reduce runoff peaks for several return periods through the use of one structure. The subroutine can model rectangular, v-notch and proportional weirs, perforated risers, emergency spillways, circular and rectangular orifices, open and grated drop inlets, discharge pipes, outfall culverts, and outfall channels. MSRM will adjust outlet capacity for riser box submergence and for inlet-outlet control in the outfall culvert.

## APPLICATIONS OF PSUHM

The software package has an interactive structure that allows it to model an infinite number of hydrologic situations. The user must decide on a sequence of hydrologic operations necessary to solve the problem at hand and then select the appropriate modules from the model to complete the task. On selection of a given module, the subprogram will prompt the user to enter the required information for completing the module task. On task completion, the user is returned to the main menu for the next task selection. This process is repeated until the sequence of desired operations is completed.

The following example will illustrate the use and interactive structure of PSUHM. This example consists of a hypothetical design based on an actual watershed and highway-stream crossing in Pennsylvania.

## PROTECTING A HIGHWAY CROSSING IN THE WHISKEY RUN WATERSHED

A highway crossing at Whiskey Run located along old Route 28 in East Franklin Township, Armstrong County, Pennsylvania, contains a 50-ft, 6- × 6-ft box culvert that was built in 1925 to control the 25-year flood. Under full-flow conditions, the culvert capacity is 440 $ft^3$/sec, assuming submerged outlet with head of 4.0 ft. The watershed area of 645 acres above the culvert is shown in Figure 2. Since 1925, the area above the culvert has developed from an agricultural watershed to a single-family residential watershed. Highway flooding at the culvert has been observed on numerous occasions. The flooding condition could be corrected by the construction of a stormwater detention facility within the contributing watershed. In addition to correcting the roadway flooding condition, the detention facility should also be designed to protect the downstream receiving channel from erosion by reducing the 5-year flood peak back to the 2-year flood peak.

Thus the 2- and 25-year events (Q2 and Q25) are chosen as the levels for stream channel and highway crossing protection. PSUHM has been used to design a detention facility that will meet release targets of Q2 and Q25 from Subarea

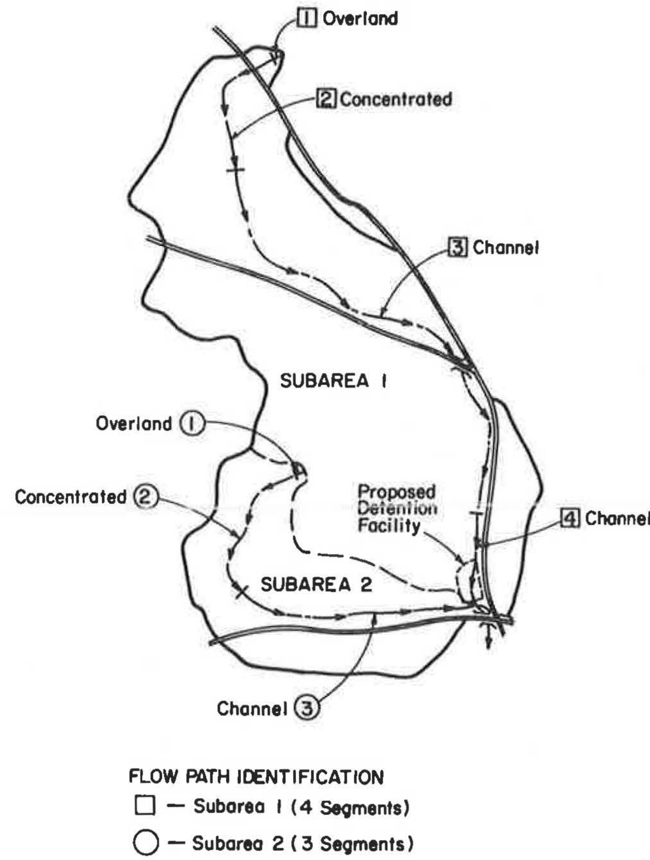



**FIGURE 2 Sketch of Whiskey Run watershed.**

1. The first step is to analyze runoff peaks and volumes for Q2, Q5, and Q25 so that the basin release targets can be set.

### Assembling Physical Data for Whiskey Run Watershed

The engineer must collect watershed physical information before using PSUHM as a runoff model. In this case involving a sizeable watershed, it is reasonable to use the SCS tabular hydrograph module in PSUHM. The necessary information includes watershed area, land use types, hydrologic soil groups (HSG) for each land use, and travel times along critical runoff flow paths.

The watershed area and land use were determined from a USGS 1:24,000 topographic map and by field survey, respectively. Figure 2 shows the land use of the Whiskey Run watershed. In the process of gathering the land use information, it is observed that the watershed should be divided into two subareas as shown in Figure 3. The upper subarea, Subarea 1 (SA1), contains all of the developed portion of the watershed. The lower subarea, Subarea 2 (SA2), is totally agricultural or wooded. This division leads to a natural division in type of land use in the two subareas and should make the results of the model more accurate. The soil type information for each land use was determined from SCS soil maps for the county.

The runoff flow paths for the watershed were also determined by field survey and a USGS 1:24,000 scale topographic

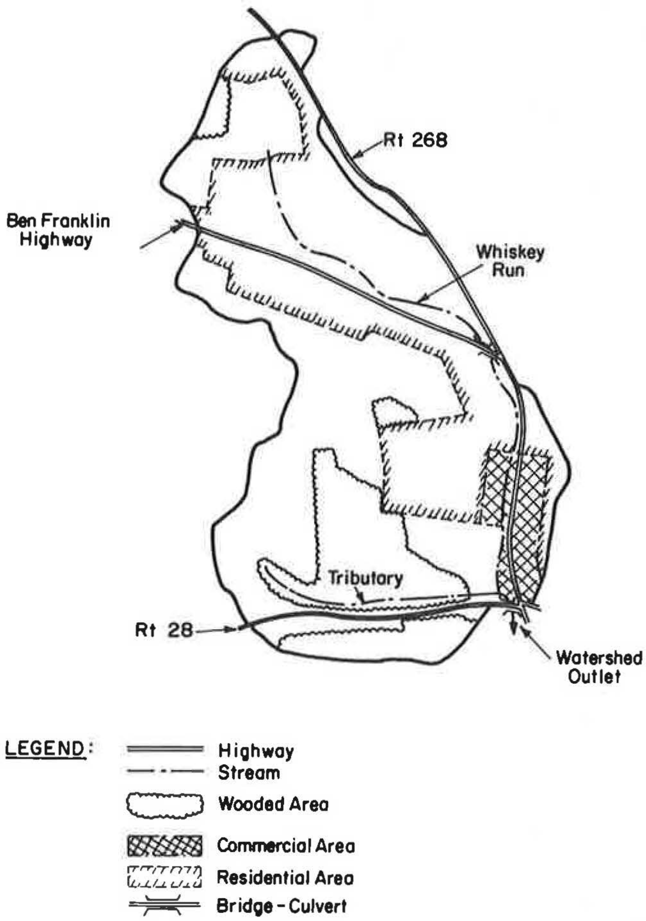



**FIGURE 3 Subarea definition and detention facility location.**

map. Several flow paths were investigated to determine which flow path was critical, that is, which flow path had the longest travel time.

### Estimating Runoff and Designing a Detention Facility

With the drainage area, land use, and travel time information, the runoff hydrographs for Whiskey Run watershed can be generated. The appropriate steps would be as follows:

Step 1. Curve Number Weighting Using Module 1A. The data shown in Tables 1 and 2 on land use, soil group, curve number (CN), and area (in acres) are entered into the program and used to determine area-weighted CN values. The results from this module are weighted CN values of 76 for SA1 and 63 for SA2, as shown in the figure.

Step 2. Travel Time Calculations Using Module 1B. Within Module 1B, the SCS segmental option is used for each subarea. The data for each subarea are in the printout of the module shown in Figure 4. In the output, $L$ = length, $S$ = slope, $P$(2-year/24-hr) = rainfall, $A$ = area, $P$ = wetted perimeter, and $n$ = Manning roughness coefficient. Overland flow is defined as sheet flow; concentrated flow is defined as swale or ditch flow; channel flow is defined as flow in a well-defined watercourse. The rainfall depth required for the overland flow calculation in each area is by definition the 2-year/24-hr rainfall depth and was determined by reference to the Pennsylvania IDF charts (*6*).

Step 3. SCS TR–55 Tabular Hydrographs Using Module 3B. With the results of the first two steps, Module 3B can calculate a runoff hydrograph for Q2, Q5, and Q25. The 24-hr rainfalls are 2.3, 2.7, and 3.7 in., respectively, from the Pennsylvania IDF charts (*6*). The printout of the module results is presented in Table 3 for the 25-year hydrograph. The peak flow is 602 $ft^3$/sec, which is 162 $ft^3$/sec greater than the capacity of the culvert. From this result, the size of the detention pond necessary to reduce the peak flow to something less than 440 $ft^3$/sec can be estimated.

Similar runs with Module 3B for the 2- and 5-year floods produce SA1 peaks of 184 and 263 $ft^3$/sec, respectively. Runoff depths for SA1 are 0.82 and 1.51 in., respectively, for Q2 and Q5.

Step 4. Sizing the Detention Facility. Figure 6 shows that the bulk of the runoff (525 of the 602 $ft^3$/sec) comes from SA1 at time 12.5 hr. It is reasonable therefore to place the detention facility in the lower end of this subarea. The Q25 basin inflow peak is 525 $ft^3$/sec, whereas the target outflow is now 440 $ft^3$/sec (culvert capacity) less the 78 $ft^3$/sec from SA2 for a target release of 362 $ft^3$/sec for Q25. At the same time, it is desired to use the detention basin in SA1 to reduce Q5 of 263 $ft^3$/sec to Q2 of 184 $ft^3$/sec to protect the receiving channel below the detention basin from excessive bank erosion. Thus, the two outflow targets from the detention facility are 184 and 362 $ft^3$/sec for the Q5 and Q25 events, respectively.

The detention basin design proceeds in three steps. First, a preliminary estimate of the volume of storage required at the SA1 site is made and a storage-elevation table is devel-

TABLE 1 PRINTOUT OF CURVE NUMBER WEIGHTING MODULE 1A FOR WHISKEY RUN WATERSHED, SUBAREA 1

| LANDUSE | SOIL GROUP | CN | AREA | % TOTAL AREA |
|---|---|---|---|---|
| MEADOW - DENSE GRASS | B | 58 | 21.0 | 4.29 |
| MEADOW - DENSE GRASS | C | 71 | 136.0 | 27.81 |
| WOODED - THICK STAND | C | 70 | 32.0 | 6.54 |
| RESIDENTIAL (1/2 A) | B | 70 | 37.0 | 7.57 |
| RESIDENTIAL (1/2 A) | C | 80 | 230.0 | 47.03 |
| COMMERCIAL | C | 94 | 33.0 | 6.75 |

TOTAL AREA = 489 acres WEIGHTED CN = 76

TABLE 2 PRINTOUT OF CURVE NUMBER WEIGHTING MODULE 1A FOR WHISKEY RUN WATERSHED, SUBAREA 2

| LANDUSE | SOIL GROUP | CN | AREA | % TOTAL AREA |
|---|---|---|---|---|
| MEADOW - DENSE GRASS | B | 58 | 67.0 | 42.95 |
| MEADOW - DENSE GRASS | C | 71 | 65.0 | 41.67 |
| WOODED - THICK STAND | B | 55 | 19.0 | 12.18 |
| WOODED - THICK STAND | C | 70 | 5.0 | 3.21 |

TOTAL AREA = 156 acres WEIGHTED CN = 63

```
*********************************************************************
* TRAVEL TIME CALCULATIONS - SCS Segmental Approach, TR-55 (1986) *
*********************************************************************

                    **  SUMMARY for SUBAREA 1  **

Segment 1: OVERLAND FLOW
     L = 50 ft, S = .015 ft/ft, n = .15, P(2yr/24hr) = 2.3 in
     Travel Time = 7.4 minutes

Segment 2: CONCENTRATED FLOW
     L = 2100 ft, S = .033 ft/ft, UNPAVED surface
     Travel time = 11.9 minutes

Segment 3: CHANNEL FLOW
     A = 10 ft^2, P = 8.66 ft, L = 6800 ft, S = .028 ft/ft, n = .035
     Travel Time = 14.5 minutes

Segment 4: CHANNEL FLOW
     A = 40 ft^2, P = 17.3 ft, L = 1300 ft, S = .014 ft/ft, n = .03
     Travel Time = 2.1 minutes

      TOTAL TRAVEL TIME for path SUBAREA 1 = 36 minutes <----


                    **  SUMMARY for SUBAREA 2  **

Segment 1: OVERLAND FLOW
     L = 50 ft, S = .025 ft/ft, n = .24, P(2yr/24hr) = 2.3 in
     Travel Time = 8.8 minutes

Segment 2: CONCENTRATED FLOW
     L = 2200 ft, S = .049 ft/ft, UNPAVED surface
     Travel time = 10.3 minutes

Segment 3: CHANNEL FLOW
     A = 3 ft^2, P = 4.82 ft, L = 3260 ft, S = .043 ft/ft, n = .04
     Travel Time = 9.7 minutes

      TOTAL TRAVEL TIME for path SUBAREA 2 = 28.8 minutes <----
```

**FIGURE 4 Printout of travel time calculations Module 1B for Whiskey Run watershed: (*top*) Subarea 1, and (*bottom*) Subarea 2.**

oped. Second, the hydraulic outlet structure is sized with appropriate stages to control Q5 and Q25 at their respective targets and a discharge-elevation table is developed. Third, storage-indication routing of Q5 and Q25 inflow hydrographs is performed through the basin to determine if the basin outflow targets with the particular combination of outlet structure geometry and basin storage are met. Repetition of this process is carried out readily using the multistage routing model in Module 4C.

A first guess of the total basin storage required to control Q25 is obtained by applying the approximate routing chart in Figure 6–1 of TR–55 (*2*). This figure contains plots of the volume ratio $V_s/V_r$ as a function of the peak flow ratio $Q_o/Q_i$, where $V_s$ is the estimated storage volume, $V_r$ and $Q_i$ are runoff volume and peak of the inflow hydrograph, and $Q_o$ is the target outflow peak. In this example, $Q_o/Q_i$ is 362/525 (0.690) for Q25. $V_s/V_r$ is then found to be 0.21 from Figure 6–1 of TR–55 (*2*). The depth of runoff in SA1 for Q25 is 1.51 in. (from Figure 6) and $V_r$ is then 489 acres × 1.51 in./12 [61.5 acre-ft (AF)]. The estimated gross storage, $V_s$, required to reduce Q25 leaving SA1 from 525 to 362 ft³/sec is thus computed as 0.21 × 61.5 AF = 12.9 AF. This estimate of storage will be revised by modified Puls routing through the trial basin. Using a detailed topographic survey for the SA1 site, storage-elevation data can be developed at 1-ft intervals as presented in Table 4.

Step 5. Sizing the Outlet Structure Using Module 4C. In the attempt to control two points on the frequency curve for SA1 of Whiskey Run, it is reasonable to assume that two stages are required in the basin outlet structure. Both stages are mounted in a riser box, which in turn discharges to an outlet culvert through the embankment to the downstream receiving channel. The lower first stage is sized to reduce Q5 from 263 to 184 ft³/sec. This outlet, together with the upper second stage, jointly reduces Q25 from 525 to 362 ft³/sec. Consequently, the upper-stage design depends on the hydraulic capacity of the lower stage and the design analysis must start there.

From TR–55 (2) and the tabular hydrograph results of Step 3, the runoff volume for the 5-year flood, $V_r$, is (0.82 in.) × (489 acres)/12 = 33.4 AF. The peak flow ratio $Q_o/Q_i$ is 184/263 (0.70), and from Figure 6–1 in TR–55 (*2*) the $V_s/V_r$ ratio is 0.21. $V_s$ is then 0.21 × 33.4 AF = 7.0 AF. According to the storage-elevation relation in Table 4, 7.0 AF of storage

TABLE 3 PRINTOUT OF TABULAR HYDROGRAPH MODULE 3B FOR 25-YEAR STORM OF WHISKEY RUN WATERSHED

```
*******************************************************************************
*  PSUHM: MODULE <3-B> - SCS TR-55 TABULAR METHOD                             *
*******************************************************************************

                         WATERSHED TITLE: WHISKEY RUN

                    25  YR. STORM: PRECIPITATION =  3.7  in.

===============================================================================

                         SUMMARY OF INPUT PARAMETERS

SUBAREA    AREA    CURVE    IA/P    RUNOFF    TC     ADJ. TC    TT     ADJ. TT
          (sqmi)   NUMBER            (in)    (hrs)    (hrs)    (hrs)    (hrs)
-------------------------------------------------------------------------------
   1      0.764     76     0.171    1.51    0.600    0.500    0.000    0.100
   2      0.244     63     0.317    0.76    0.480    0.500    0.000    0.000
-------------------------------------------------------------------------------
COMPOSITE 1.008     73              1.33

                   INDIVIDUAL SUBAREA & COMPOSITE HYDROGRAPHS

SUBAREA                             TIME (hrs)
          11.0  11.9  12.2  12.5  12.8  13.2  13.6  14.0  15.0  17.0  20.0  26.0
-------------------------------------------------------------------------------
   1        12    38   203   525   333   142    86    65    44    29    19     0
   2         0     0    27    78    43    22    15    12     9     6     4     0
-------------------------------------------------------------------------------
COMPOSITE   12    38   230   602   376   164   101    77    53    35    24     0

              THE PEAK FLOW IS  602.4 cfs - OCCURS AT 12.5 hrs

===============================================================================
```

TABLE 4 STORAGE-ELEVATION DATA FOR WHISKEY RUN DETENTION FACILITY

| Elevation (ft) | Storage (acre-ft) |
|---|---|
| 998 | 0 |
| 999 | 0.23 |
| 1000 | 0.94 |
| 1001 | 2.18 |
| 1002 | 3.95 |
| 1003 | 6.04 |
| 1004 | 8.28 |
| 1005 | 10.67 |
| 1006 | 13.22 |
| 1007 | 15.50 |

occurs at elevation 1003.44 ft. Because the floor elevation is 998.00 ft, there will be approximately 5.44 ft of water (head) impounded in the detention basin during maximum stage of the Q5 event. Using the FHWA culvert charts (*10*), it is determined that two 48-in.-diameter circular orifices with invert elevations at 998.00 ft will discharge just slightly more than 184 ft$^3$/sec at a maximum headwater of 4.38 ft. The corresponding water surface elevation is 1002.38 ft and the associated storage is 4.75 AF. Preliminary runs with the outlet rating and basin routing programs in the multistage routing Module 4C reveal that maximum outflow will be 187 ft$^3$/sec and that 4.75 AF of storage are required to control Q5. These numbers must be confirmed by actual routing, as described later.

In the second stage, calculation indicates that reducing Q25 from 525 to 362 ft$^3$/sec will require storage of approximately 12.9 AF. According to Table 4, this storage occurs at elevation 1005.88 ft. Thus, the maximum stage will be 7.88 ft above the basin floor elevation of 998.00 ft. According to the preliminary rating curve developed in Module 4C, the discharge through the lower stage outlets will be 294 ft$^3$/sec at elevation 1005.88 ft. The second stage must therefore release 68 ft$^3$/sec at elevation 1005.88 ft under Q25 inflows. The invert or bottom of the upper stage will be located just above the maximum water surface elevation reached during Q5, say 1002.40 ft.

TABLE 5 PRINTOUT OF ROUTING OUTPUT FROM MSRM MODULE 4C FOR PROPOSED TWO-STAGE OUTLET POND FOR WHISKEY RUN WATERSHED

| TIME hrs. | HYDROGRAPH INFLOW,cfs | BASIN INFLOW,cfs | STORAGE acre-ft | ELEVATION ft.ABOVE msl | BASIN OUTFLOW,cfs | OUTFLOW TOTAL,cfs | RETENTION TIME,hrs |
|---|---|---|---|---|---|---|---|
| 0.00 | 12.00 | 12.00 | 0.000 | 998.00 | 0.00 | 0.00 | 0.00 |
| 0.17 | 14.40 | 14.40 | 0.145 | 998.63 | 5.37 | 5.37 | 0.18 |
| 0.33 | 17.07 | 17.07 | 0.237 | 999.01 | 12.77 | 12.77 | 0.19 |
| 0.50 | 20.40 | 20.40 | 0.302 | 999.10 | 15.20 | 15.20 | 0.21 |
| 0.67 | 25.87 | 25.87 | 0.389 | 999.22 | 18.43 | 18.43 | 0.21 |
| 0.83 | 34.63 | 34.63 | 0.514 | 999.40 | 23.88 | 23.88 | 0.21 |
| 1.00 | 60.20 | 60.20 | 0.756 | 999.74 | 35.89 | 35.89 | 0.19 |
| 1.17 | 171.20 | 171.20 | 1.599 | 1000.53 | 73.10 | 73.10 | 0.16 |
| 1.33 | 382.80 | 382.80 | 3.893 | 1001.97 | 147.84 | 147.84 | 0.18 |
| 1.50 | 524.60 | 524.60 | 7.493 | 1003.85 | 236.81 | 236.81 | 0.24 |
| 1.67 | 441.93 | 441.93 | 10.092 | 1004.76 | 352.36 | 352.36 | 0.31 |
| 1.83 | 312.27 | 312.27 | 10.381 | 1004.88 | 359.82 | 359.82 | 0.37 |
| 2.00 | 210.00 | 210.00 | 9.250 | 1004.41 | 326.75 | 326.75 | 0.42 |
| 2.17 | 153.27 | 153.27 | 7.769 | 1003.77 | 251.46 | 251.46 | 0.46 |
| 2.33 | 117.90 | 117.90 | 6.470 | 1003.19 | 208.29 | 208.29 | 0.48 |
| 2.50 | 96.00 | 96.00 | 5.266 | 1002.63 | 180.51 | 180.51 | 0.46 |
| 2.67 | 81.83 | 81.83 | 4.181 | 1002.11 | 154.89 | 154.89 | 0.43 |
| 2.83 | 71.97 | 71.97 | 3.273 | 1001.62 | 130.70 | 130.70 | 0.39 |
| 3.00 | 65.20 | 65.20 | 2.553 | 1001.21 | 111.04 | 111.04 | 0.35 |
| 3.17 | 60.03 | 60.03 | 2.018 | 1000.87 | 91.89 | 91.89 | 0.32 |
| 3.33 | 55.07 | 55.07 | 1.656 | 1000.58 | 75.68 | 75.68 | 0.31 |
| 3.50 | 51.00 | 51.00 | 1.416 | 1000.38 | 65.34 | 65.34 | 0.29 |
| 3.67 | 47.77 | 47.77 | 1.245 | 1000.25 | 58.23 | 58.23 | 0.28 |
| 3.83 | 45.80 | 45.80 | 1.120 | 1000.15 | 53.50 | 53.50 | 0.27 |
| 4.00 | 43.80 | 43.80 | 1.025 | 1000.07 | 49.91 | 49.91 | 0.26 |
| 4.17 | 42.30 | 42.30 | 0.950 | 1000.01 | 47.08 | 47.08 | 0.26 |
| 4.33 | 40.77 | 40.77 | 0.894 | 999.94 | 44.02 | 44.02 | 0.26 |
| 4.50 | 39.20 | 39.20 | 0.855 | 999.88 | 41.69 | 41.69 | 0.26 |
| 4.67 | 37.93 | 37.93 | 0.824 | 999.84 | 39.88 | 39.88 | 0.26 |
| 4.83 | 36.63 | 36.63 | 0.799 | 999.80 | 38.38 | 38.38 | 0.26 |
| 5.00 | 35.30 | 35.30 | 0.775 | 999.77 | 36.99 | 36.99 | 0.26 |
| 5.17 | 34.07 | 34.07 | 0.752 | 999.74 | 35.71 | 35.71 | 0.26 |
| 5.33 | 32.73 | 32.73 | 0.729 | 999.70 | 34.49 | 34.49 | 0.26 |
| 5.50 | 31.50 | 31.50 | 0.705 | 999.67 | 33.24 | 33.24 | 0.26 |
| 5.67 | 10.30 | 10.30 | 0.579 | 999.49 | 26.78 | 26.78 | 0.38 |

TOTAL ROUTING MASS BALANCE DISCREPANCY= .06 %

PEAK INFLOW = 524.60 cfs. PEAK OUTFLOW = 359.82 cfs.

ROUTING TIME STEP = 0.17 hours NUMBER OF OUTFLOW HYDROGRAPH POINTS = 34

MODIFIED PULS BASIN ROUTING FOR ---25 YR

INFLOW HYDROGRAPH FILENAME: B:WHIS125.HYD
BASIN STORAGE/ELEVATION DATA FILENAME: B:WHISA1.ES
OUTLET STRUCTURE DISCHARGE/ELEVATION DATA FILENAME: B:TRIPNU2.EO

The head on the second stage will then be 3.48 ft during Q25. A rectangular orifice 2 ft high by 7 ft long installed vertically above the twin 48-in. circular orifices would be satisfactory here. However, it was decided to use a grated drop inlet with an effective area of 14 ft$^2$ and effective perimeter of 14 ft installed on the top of the riser box at elevation 1002.40 ft. Preliminary analysis with Module 4C indicates that Stages 2 and 1 together perform satisfactorily in meeting the Q25 outflow target of 362 ft$^3$/sec. Again, full routing with the modified Puls routine in Module 4C is required for confirmation of this preliminary outlet design.

The final element in the basin outlet structure is the outfall culvert leading from the concrete riser box upstream of the embankment to the lower receiving channel downstream from the embankment. The discharge rating curve for the outlet structure must include a correction for submergence of the riser box by the outfall culvert. Inspection of the site near the embankment and present stream location indicates that a culvert length of 50 ft and slope of 0.005 ft/ft, with upstream invert at elevation 995.00 ft, will be required. Normal depth tailwater would not be expected to submerge the culvert at its lower end. The culvert will be mounted in a square-end headwall just downstream from the riser box and consist of corrugated metal pipe with a Manning $n$ value of 0.012. From the FHWA culvert charts (*10*) or the discharge pipe option in Module 4C, an 8.5-ft corrugated metal pipe (CMP) culvert

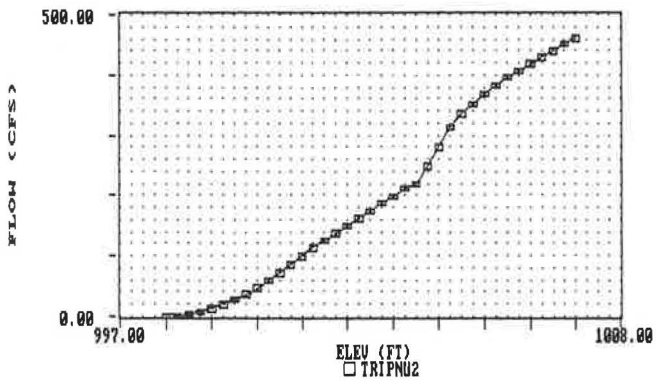



**FIGURE 5 Screen dump of graphics display Module 1D of the outlet rating curve for Whiskey Run Subarea 1 detention facility.**

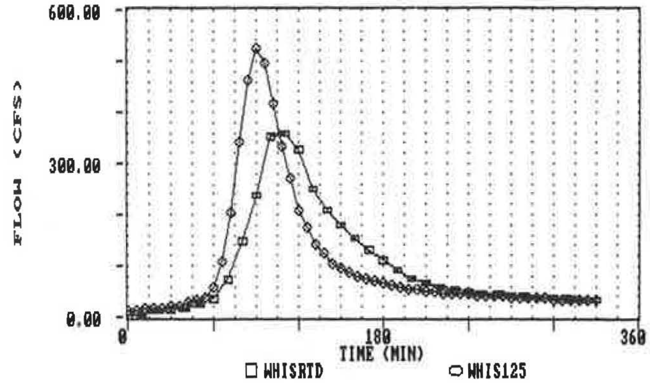



**FIGURE 6 Screen dump of graphics display Module 1D of the Q25 routed hydrograph for Whiskey Run Subarea 1.**

will discharge a flow of 478 ft³/sec under inlet control at elevation 1005.80 ft. This flow is greater than the riser inlet capacity at this elevation. In general, the outfall culvert should be sufficiently large that it does not control pond releases through the two-stage riser box. Although the riser box can still be submerged, hydraulic control of the detention pond is maintained by the riser inlets, rather than by the outfall culvert. Submergence of the riser box inlets is an important factor because it can significantly reduce basin outflow capacity. Module 4C has the ability to compute the outfall culvert capacity under both inlet and outlet control and to make appropriate corrections for submergence of the riser box inlets. Final confirmation of the preliminary design is conducted by routing Q5 and Q25 through the basin to determine whether the outflow targets of 184 and 362 ft³/sec are met.

Step 6. Routing of Q5 and Q25 Through the Trial Detention Structure Using Module 4C. The basin routing operation contained in Module 4C of PSUHM can test the two-stage outlet structure proposed for the SA1 site. Module 4C combines the storage-elevation data in Table 4 and the outlet discharge rating curve previously developed and computes a routed outflow hydrograph by the modified Puls method. Module 4C prompts for the names of the inflow hydrograph file, the storage-elevation file, and the outflow-elevation file. It asks for the starting water surface in the detention basin and the time step for the routing operation. Output takes the form of a routed outflow hydrograph for the Q25 event. Inspection of the basin outflow hydrograph reveals that the maximum outflow is only 322 ft³/sec and that the corresponding storage used is 11.61 AF. Similar execution of Module 4C for the Q5 event produces a maximum outflow of 167 ft³/sec at an elevation of 1002.70 ft. This routing operation confirms that the first stage of the two-stage riser is reasonably effective in meeting the target Q5 outflow of 184 ft³/sec, but that the second-stage drop inlet should be enlarged to meet the 362-ft³/sec target of Q25. This was accomplished by enlarging the effective area to 20 ft² and the effective perimeter to 16 ft. Note that the top of the grated inlet has been raised to 1002.70 ft. As shown in the Module 4C output routing table in Table 5, Q25 through the revised outlet structure now produces a maximum outflow of 360 ft³/sec and storage volume of 10.4 AF at elevation 1004.88 ft. This is significantly less than the 12.9 AF projected initially for the site and demonstrates the clear need to perform rigorous hydraulic routing calculations

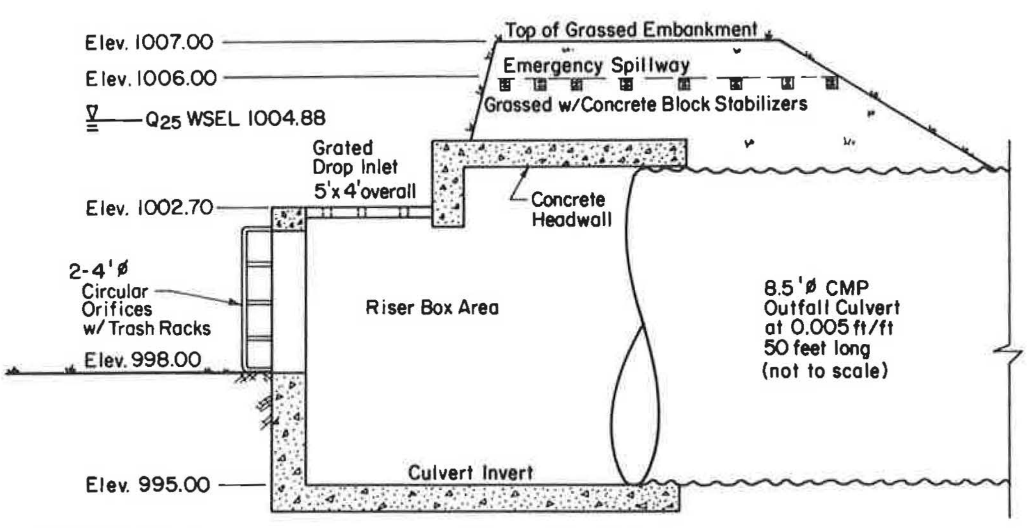



**FIGURE 7 Sketch of the two-stage outlet structure of the detention basin.**

for larger detention structures. Plots of the final outlet rating curve and the routed Q25 hydrograph are shown in Figures 5 and 6, respectively. A sketch of the final outlet structure is shown in Figure 7.

As a matter of interest, Q25 was routed through the final detention basin with the upper stage deleted from the outlet structure. As expected, maximum outflow was considerably reduced to 256 $ft^3$/sec versus the target 362 $ft^3$/sec. On the other hand, maximum storage required was increased to 13.7 AF at elevation 1006.22 ft. This example illustrates the difficulty of meeting multiple outflow targets with a single-stage outflow structure. The final two-stage structure in this case does an excellent job of meeting both the Q5 and the Q25 targets and probably provides considerable protection at other design frequencies as well.

## REFERENCES


1. D. F. Kibler, E. L. White, and T. A. Seybert. *Users Manual to the Penn State Urban Hydrology Model*. Pennsylvania State University, Department of Civil Engineering, May 1990.
2. *Urban Hydrology for Small Watersheds*. Technical Release 55, U.S. Department of Agriculture, Soil Conservation Service, June 1986.
3. *Urban Hydrology for Small Watersheds*. Technical Release 55, U.S. Department of Agriculture, Soil Conservation Service, Jan. 1975.
4. D. F. Kibler, ed. *Urban Stormwater Hydrology*. Water Resources Monograph 7, American Geophysical Union, Washington, D.C., 1982.
5. D. L. Yarnell. *Rainfall Intensity-Frequency Data*. Miscellaneous Publication 204, U.S. Department of Agriculture, Washington, D.C., 1935.
6. G. Aron, D. J. Wall, E. L. White, et al. *Pennsylvania Department of Transportation Storm Intensity-Duration-Frequency Charts (PDT–IDF)*. Report FHWA-PA-85-032. Environmental Resources Research Institute (Institute for Research on Land and Water Resources), Pennsylvania State University, University Park, Pa., May 1986.
7. *Hydrology*. National Engineering Handbook, Section 4, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 1972.
8. W. Viessman, J. W. Knapp, and G. L. Lewis. *Introduction to Hydrology*. 3rd ed., Harper and Row, New York, 1989.
9. A. S. Chamberlain III. *A Multiple Stage Detention Basin Analysis and Design Model*. M.S. thesis, Department of Civil Engineering, Pennsylvania State University, University Park, Pa., Aug. 1986.
10. *Hydraulic Design of Highway Culverts*. Report FHWA-IP-85-15. Hydraulic Design Series No. 5, FHWA, U.S. Department of Transportation, Sept. 1985.


*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# Reduction of Salt Buildup and Twig Injury in Roadside Peach Trees with Film-Forming Sprays

CALVIN CHONG AND GLEN P. LUMIS

Peach trees located along a major highway were sprayed in November 1987 with five film-forming products (Folicote, RD 1725, RD 1726, Rhodorsil, and Joncryl 1938). There were also two control (unsprayed and burlap-covered) treatments. By spring 1988, burlapped twigs accumulated the least chloride (0.29 percent by dry weight of twig tissue) and twigs were least injured (5.3 cm dieback). In contrast, unsprayed twigs accumulated the most chloride (1.79 percent) and were the most injured (139 cm dieback). Corresponding data for spray-treated twigs were intermediate, indicating small-to-moderate beneficial influence of most products, especially RD 1726 (1.39 percent chloride ion; 8.7 cm dieback) and Joncryl 1938 (1.24 percent chloride ion; 7.4 cm dieback). Burlapped and unsprayed twigs contained 0.08 and 0.31 percent sodium, respectively, and all spray-treated twigs between 0.30 and 0.36 percent sodium. In another study, peach trees were sprayed with Folicote, RD 1725, and four other emulsion-based formulations (RD 2033, RD 2034, RD 2035, and RD 2036); half of each tree was sprayed several times during the winter with a 2 percent rock salt solution. Twigs treated with RD 2034 showed the least injury and accumulated moderately less salt than the control twigs from both salted and nonsalted sides of trees. All other treatments were ineffective, and in fact twigs treated with RD 2034 accumulated more salt than the control and had the greatest injury. Scanning electron and light microscopy revealed progressive deterioration in the surface integrity of Folicote, RD 1725, RD 2033, RD 2034, RD 2035, and RD 2036 during the winter. RD 2033 deteriorated the least and RD 2034 the most.

Application of salt to major highways during the winter is a common practice in northern areas of the United States and Canada. Numerous studies have established that roadside salt sprays cause significant damage to roadside vegetation (*1*,*2*). Expression of salt injury symptoms results from physiological drought or desiccation (*3*,*4*).

Film-forming antitranspirants or antidesiccants have been used experimentally to prevent winter injury (*4–6*). These products form a thin vapor barrier that limits water loss (*7*).

Reisch (*8*) reported that the antidesiccants Vapor Gard and Foli-Gard reduced needle browning on white pine and Norway spruce sprayed with sodium chloride (NaCl). Emmons et al. (*9*) reported that, as in Germany, antidesiccants were ineffective in reducing salt damage under Minnesota highway conditions. In fact, both Vapor Gard and Wilt Pruf increased damage on Austrian pine; Wilt Pruf caused heavy needle mortality of Pfitzer juniper (*9*). In view of the limited research, inconsistent results, and renewed interest in the potential of film-forming products as aids to protecting trees against roadside salt, Piedrahita (*10*) screened 18 different antidesiccants and related film-forming products during a 3-year period and identified five superior products for further testing.

The objectives of this study were to reevaluate under field conditions these five previously selected products, to test four other formulations, and to assess the durability or integrity of these new products during the winter using microscopy.

C. Chong, Ministry of Agriculture and Food, Horticultural Research Institute of Ontario, Vineland Station, Ontario, Canada L0R 2E0. G. P. Lumis, Department of Horticulture Science, University of Guelph, Guelph, Ontario, Canada N1G 2W1.

## MATERIALS AND METHODS

### Plant Material and Film Treatments

#### *Location 1*

Twenty 8-year-old trees of Garnet Beauty peach [*Prunus persica* (L.) Batsch.] planted in two 10-tree rows bordering the southern side of a major highway near Grimsby, Ontario, were used. Row 2 was located 36 m away and furthest from the highway; Row 1 was located 31 m away from the highway and separated from Row 2 by a row of pear trees. There was a buffer row of pear trees in front of Row 1 (26 m from the highway).

On November 9, 1987, selected branches on each tree were sprayed with Folicote, RD 1725, RD 1726, Rhodorsil, and Joncryl 1938, prepared according to the manufacturers' recommended dilution rates (Table 1). There were also two control (unsprayed and burlap-covered) treatments. During spraying, the prevailing weather was clear and sunny, and air temperature was 10°C. Each formulation was applied to the point of runoff with a 20-L compressed-air sprayer to which was added Triton XR surfactant (Rohm and Haas, Ontario) at a rate of 2.5 ml per liter of product. Overspray onto adjacent branches was prevented by a large polyethylene sheet covering. The experimental design was split plot with treatments as main plots and rows as subplots.

#### *Twig Samples*

On April 11, 1988, two 30-cm twigs per treatment branch were sampled for chemical analysis. On May 12 (budbreak), dieback was determined on two similar twigs and expressed as the distance in centimeters from the terminal to the first

TABLE 1 RATES AND DESCRIPTION OF FILM-FORMING PRODUCTS

| Product | Type of Product | Major Use | Dilution $(v/v)^z$ | % Solids at dilution | Low temperature threshold (o°C) | Colour after application | Source |
|---|---|---|---|---|---|---|---|
| Folicote | Wax emulsion | Antidesiccant | 1:10 | 5.0 | 6-8 | Translucent | Aquatrols Corp., N.J. |
| Joncryl 1938 | Acrylic polymer | Water-proof | 1:5 | 10.0 | 4-5 | " | Johnson and Sons, Ontario |
| Rhodorsil (Siliconate 51T) | Silicon | Treatment of tile and brick | 1:50 | 8.0 | 0 | " | May and Baker, Ontario |
| RD 1725 | Wax emulsion | Tree protectant[y] | 1:5 | 10.0 | 4-5 | " | International Waxes, Ontario |
| RD 1726 | " " | " | 1:3 | 10.0 | 4-5 | " | " " |
| RD 2033 | " " | " | 1:3 | 10.6 | 4-5 | White | " " |
| RD 2034 | " " | " | 1:3 | 12.0 | 4-5 | " | " " |
| RD 2035 | " " | " | 1:3 | 11.7 | 4-5 | " | " " |
| RD 2036 | " " | " | 1:5 | 10.0 | 4-5 | Translucent | " " |

[y] New product formulated for this research project.

[z] Volume of water to volume of product.

TABLE 2 DIEBACK AND SALT CONTENT IN TWIGS OF GARNET BEAUTY PEACH TREES SPRAYED WITH VARIOUS FILM-FORMING PRODUCTS

| Treatments | Twig dieback (cm)[y] | $Na^+$ (% dry weight) | $Cl^-$ (% dry weight) |
|---|---|---|---|
| | Film effects | | |
| Untreated | 13.6 a[x] | 0.31 b | 1.79 a |
| Folicote | 10.0 abc | 0.31 b | 1.45 b |
| RD 1725 | 11.8 ab | 0.32 ab | 1.35 b |
| Rd 1726 | 8.7 bcd | 0.34 ab | 1.39 b |
| Rhodorsil | 12.3 a | 0.36 a | 1.43 b |
| Joncryl | 7.4 cd | 0.30 b | 1.24 b |
| Burlap | 5.3 d | 0.08 c | 0.29 c |
| | Row effects | | |
| Row 1[v] | 13.2 a | 0.32 a | 1.48 a |
| Row 2[v] | 7.5 b | 0.25 b | 1.08 b |

Background levels: 0.035 ppm $Na^+$; 0.14 ppm $Cl^-$.

[y] Analysis of variance conducted on values transformed to log (x+1).

[x] Values in columns followed by the same letters are not significantly different from each other at the 5% level by Duncan's multiple range test.

[v] 31 and 36 m from the highway, respectively.

istic hollows formed by other films (Figures 2a–2f). Whereas RD 2035 tended to be smooth and less rippled (Figure 2d), RD 2033 (Figure 2e) and Folicote (Figure 2f) appeared more textured, especially at higher magnifications. Lenticels occluded by RD 2033 (Figure 2e) and occluded to a lesser degree by RD 2036 (Figure 2b) were typically less hollowed and more difficult to discern, indicating thorough coverage of these films.

Unprotected lenticels of nonsalted control twigs appeared wart-like at lower magnifications (Figure 3a). Corresponding salted control twigs appeared pitted (Figure 3c) and, at higher magnifications, the contour in the vicinity of these lenticels tended to appear somewhat softer or less textured (Figure 3d) than that of nonsalted counterparts (Figure 3b).

By the end of December, the surfaces of RD 2034 and RD 2035 (both nonsalted and salted twigs) under light microscope appeared glossier and thinner than in early December. Under SEM, about 25 and 10 percent of lenticels on the surfaces of RD 2034 and RD 2035, respectively, were unplugged. Salted surfaces of twigs treated with RD 2034 (Figures 4a and 4b) and RD 2035 (Figures 4c and 4d) were characterized by sunken and darkened, blotched, or watermarked areas in the vicinity of the lenticels, although most lenticels remained blocked. This appearance provides evidence of early deterioration in the surfaces of RD 2034 and RD 2035.

By late January, progressive deterioration occurring most rapidly in RD 2034 was observed in all films except RD 2033. By late March, all films, including RD 2033, appeared markedly different (Figures 5 and 6) than in December (Figure 2). Both nonsalted (Figure 5) and salted (Figure 6) films appeared thinner, etched, or weathered, with bare patches or uneven coverage. Except for RD 2033 (nonsalted, Figure 5E; salted, Figure 6E), 50 to 75 percent of lenticels were unplugged in most of the other treatments.

For apple trees, in similar SEM studies conducted by Piedrahita (*10*), Folicote, RD 1725, RD 1726, and Joncryl 1938 covered twigs evenly in all areas except the lenticels, which remained only partially plugged. By February or March, most of the lenticels were exposed. Longitudinal fissures, observed living flower bud subtended by living stem. Data for twig dieback were transformed to log ($x$ + 1) before analysis of variance.

### Location 2

Twenty-eight 12-year-old Madison peach trees, situated 45 to 50 m from the northern edge of the same highway in St. Catharines, Ontario, were assigned to four replications of seven trees each.

On November 30, 1987, individual trees within a replicate were sprayed with Folicote, RD 1725, RD 2033, RD 2034, RD 2035, or RD 2036 (Table 1). One control tree per replicate was not sprayed. During spraying, the prevailing weather was cloudy, overcast, and calm, and the air temperature was 7°C. Two hours after spraying was completed, a light drizzle started and continued for 24 hr. On December 14, 1987, January 12, 1988, February 9, 1988, and March 8, 1988, half of each tree was sprayed with a 2 percent rock salt (NaCl) solution. The experimental design was a split plot with film treatments as main plots, and nonsalted or salted sides as subplots.

### Twig Samples

Samples consisting of four 30-cm twigs were taken both from unsalted and salted sides of each tree on December 29, 1987, January 27, 1988, February 23, 1988, March 24, 1988, and April 13, 1988, for microscopic examination and chemical analysis. On May 1, all flower and vegetative buds were counted in situ on three twigs per treatment. On May 10 (budbreak), viable buds were recounted and percentages of dead flower buds per twig were calculated. Twig dieback was determined as described. Data for twig dieback and percentage of dead flower buds were transformed to log ($x$ + 1) and arcsin $\sqrt{x}$, respectively, before analysis of variance.

### Microscopy

On each sampling date, two twigs per treatment within two randomly selected replications were examined along their entire length under a stereo light microscope to assess the integrity of each film as the winter progressed. Several 1- to 2-cm fingernail scrapes along the twig gave an indication of the relative thickness of each treatment layer. A 1-cm specimen also was removed from each twig, at one-third the distance from the tip. Each specimen was mounted on an aluminum stub, air-dried for 72 hr, coated with gold-palladium, and examined at 20- to 25-mm working distance at 20 kV with a Hitachi S-570 scanning electron microscope (SEM) at four separate points, one on each side and two along the top of the specimen.

### Chemical Analysis

Twig samples for chemical analysis were dried and ground. After dry ashing at 550°C, sodium ($Na^+$) was determined by emission spectrometry and chloride ($Cl^-$) using a $Cl^-$ electrode (*11*). $Na^+$ and $Cl^-$ background levels in twigs at both locations were determined from twig samples taken in November before the start of the experiments. The amounts of $Na^+$ and $Cl^-$ were converted to percentages of the dry weight of the twig sample.

## RESULTS AND DISCUSSION

### Location 1

In the spring, Garnet Beauty peach trees closer to the highway (Row 1) had more twig dieback and accumulated more $Na^+$ and $Cl^-$ than trees further from the highway (Table 2), similar to results of other researchers (*11,12*).

Untreated twigs accumulated large quantities of $Cl^-$ (1.79 percent dry weight) and showed considerable damage (13.6-cm dieback) (Table 2). In contrast, burlapped twigs accumulated only 0.29 percent $Cl^-$ and showed considerably less damage (5.3-cm dieback). All spray-treated twigs accumulated moderately less $Cl^-$ (1.24 to 1.45 percent) and exhibited intermediate injury. Among spray treatments, twigs sprayed with RD 1726 and Joncryl had less dieback (8.7 and 7.4 cm, respectively). Accumulation of $Na^+$ was least in burlapped twigs (0.08 percent), high in Rhodorsil-treated twigs (0.36 percent), and in intermediate amounts (0.30 to 0.31 percent) in all other treatments, including untreated twigs.

### Location 2

The $Na^+$ and $Cl^-$ contents (Figure 1a) in twigs of Madison peach increased progressively during the winter, peaking or plateauing in late March. Twigs sprayed with RD 2033 accumulated the least quantities of $Na^+$ and $Cl^-$ during the winter (Figures 1a and 1b) and tended to show the least dieback (4.1 cm) and dead flower buds (13 percent) (Table 3). Compared with nonsalted twigs, salted twigs accumulated significantly more $Na^+$ and $Cl^-$ during the winter (Figures 1c and 1d), but contents apparently were not high enough to cause a difference in twig or flower bud injury (Table 3). There was no interaction between film and rock salt. Except for RD 2034, which accumulated more $Na^+$ and $Cl^-$ than control twigs (Figures 1a and 1b), and had extensive twig injury (Table 3), reductions of salt content or injury by other films were not sufficiently large to be meaningful.

### Film Integrity

Notwithstanding the drizzle that began soon after spraying was completed (on November 9, 1987), all formulations exhibited good coverage when twigs were examined 2 days later under a light microscope. There were apparent variations in surface morphology and thicknesses of the films, with RD 2033 appearing the thickest.

Under SEM, RD 1725 (Figure 2a) and RD 2030 (Figure 2b) appeared as heavily coated and velvety or smooth surfaces, with complete occlusion of the lenticels. Lenticular occlusions by RD 2034 often appeared as pebbly, flattened, and undefined areas (Figure 2c) in contrast to the character-

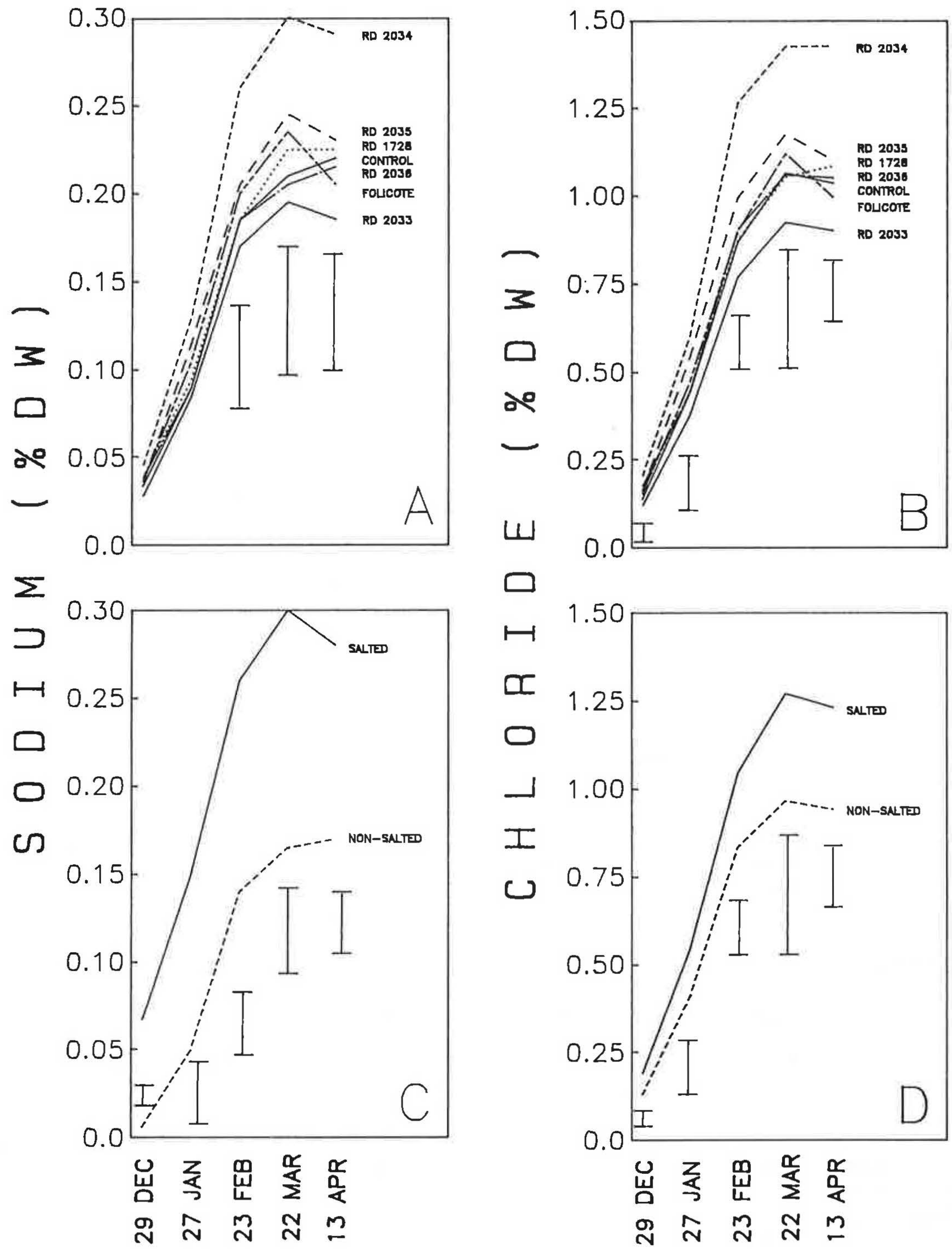



**FIGURE 1** Contents of $Na^+$ and $Cl^-$ during the winter in nonsalted and salted peach twigs covered with various film-forming products. Vertical bars represent least significant difference values at the 5 percent level of probability.

TABLE 3 DIEBACK AND PERCENTAGE OF DEAD FLOWER BUDS IN TWIGS OF MADISON PEACH TREES SPRAYED WITH VARIOUS FILM-FORMING PRODUCTS AND WITH REPEATED APPLICATIONS OF 2 PERCENT ROCK SALT SOLUTION

| Treatments | Twig dieback (cm)[z] | % Dead flower buds[y] |
|---|---|---|
| | Film effects | |
| No film | 6.0 bc[x] | 46 ab |
| Folicote | 5.9 bc | 40 ab |
| RD 1725 | 11.0 ab | 33 bc |
| RD 2033 | 4.1 c | 13 c |
| RD 2034 | 7.9 abc | 58 a |
| RD 2035 | 8.2 abc | 45 ab |
| RD 2036 | 16.5 a | 26 bc |
| | Rock salt effects | |
| Non-salted | 7.5 a | 32 a |
| Salted | 9.6 a | 38 a |

[z] Analysis of variance conducted on values transformed to log (x+1).

[y] Analysis of variance conducted on values transformed to arcsin √x.

[x] Values in columns followed by the same letter are not significantly different from each other at the 5% level by Duncan's multiple range test.

as early as mid-December in certain films and plentiful by February or March (*10*), were seldom observed and appeared only in RD 2035 and once with Folicote in the present study.

## GENERAL DISCUSSION

The field investigation conducted at Location 1 during the winter of 1987–1988 showed that all five products, Folicote, RD 1725, RD 1726, Rhodorsil, and Joncryl, moderately reduced buildup of $Na^+$ and $Cl^-$ in peach twigs. There was a tendency for less dieback in all spray-treated twigs, but only those treated with RD 1726 and Joncryl 1938 had significantly less damage statistically ($p < 0.05$) than the unsprayed control. The burlap method of protection produced excellent results as exemplified by the dramatic reduction both in salt buildup and twig injury; it has been used on a small scale for years at Location 1. However, this method is not practical on a large scale.

In investigations conducted during the winter of 1986–1987 (*10*), several or all five of these products reduced salt and twig injury to white pine, red pine, and Austrian pine, but had little or no effect on peach. There was some prevention of injury on apple trees by one or more of these products, but results were more variable or treatments ineffective after treating various other deciduous species, including silver maple, lilac, and black willow (*10*). In related tests conducted on these same species during the winter of 1987–1988, all five products resulted in small but significant reductions in twig injury of red pine. Except for Rhodorsil, all spray-treated red pine twigs also contained less salt than unsprayed twigs. There were few or no consistent effects of the products on the other species.

The investigation conducted at Location 2 indicated that film thickness and resistance to weathering play an important role. Electron microscopy indicated a progressive deterioration of surface films during the winter, and this process was apparently accentuated by salt. One product, RD 2033, which deteriorated the least and was thickest, suppressed salt levels and reduced flower bud injury by providing persistent coverage throughout the winter. In contrast, RD 2034 had the opposite effect because of early and more rapid loss of physical integrity of this film. The increases in salt content in twigs despite the film barrier suggests that the film may have reacted chemically with the salt and influenced the physiology of the twigs. According to Simini and Leone (*13*), the physical and chemical properties both of salt particles and plant tissue can be altered by high relative humidity. However, differences in films are most likely related to prevailing environmental factors (*7*) and also to physical factors of the films such as composition, plasticity, or drying characteristics (*4*). For instance,

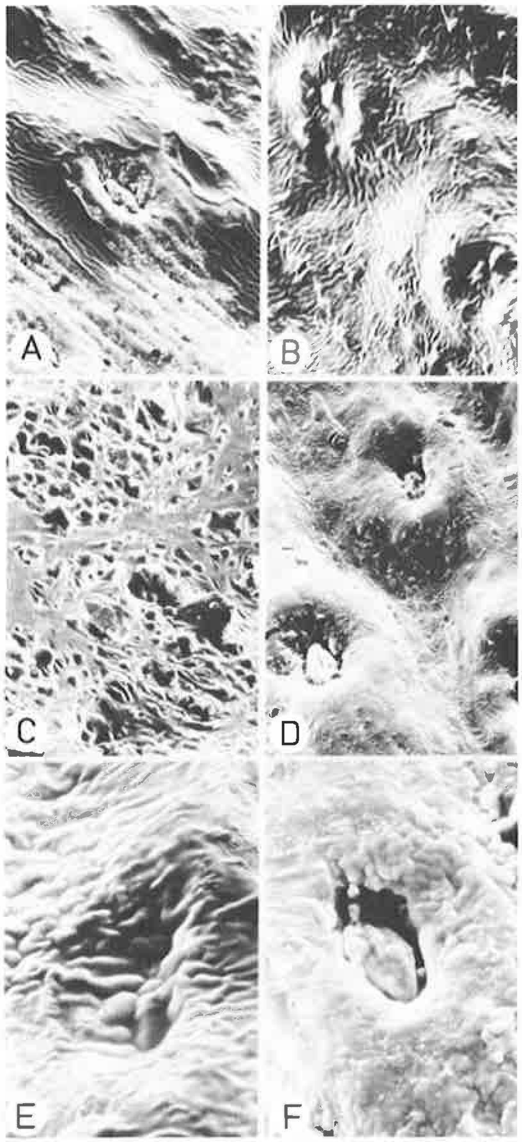



**FIGURE 2 Surfaces of peach twigs 2 days after treatment in the fall with various film-forming products: (a) RD 1725 (×1,750); (b) RD 2036 (×1,750); (c) RD 2034 (×2,500); (d) RD 2035 (×2,500); (e) RD 2033 (×5,000); (f) Folicote (×5,000).**

RD 1725 and RD 2036 are similar in composition; RD 2033 contains a different emulsifier than those of the other RD series; and RD 2034 is made of the most flexible and tacky plastic wax base. According to Davenport (*14*), phytotoxicity of film-forming products is usually caused by the emulsifier system rather than by the water vapor barrier.

## CONCLUSION

The present study confirms that film-forming products reduce buildup of salt in twigs of roadside trees and provide some protection against salt spray injury. Although the results are interesting and significant, the products have not yet been tested under severe winter conditions. Winter temperatures during the past several years at the test locations have been

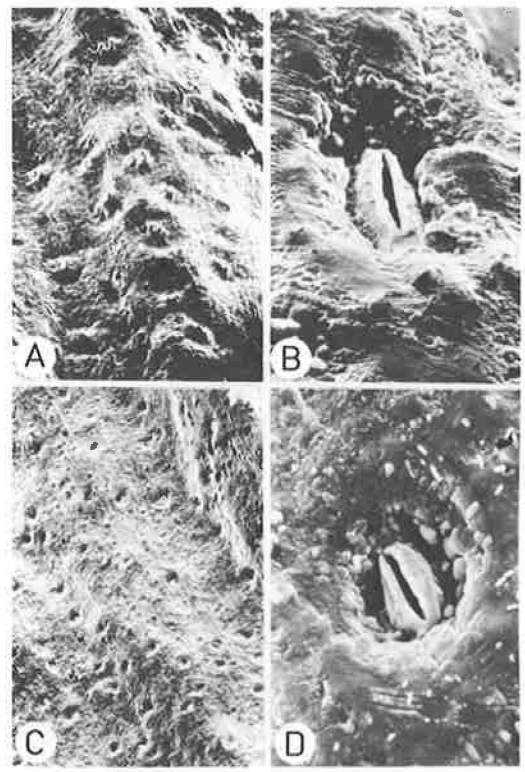



**FIGURE 3 Surfaces of nonsalted control peach twigs in late December (a) ×875, (b) ×7,500; and corresponding twigs salted with 1 application of 2 percent rock salt solution (c) ×500, (d) ×5,000.**

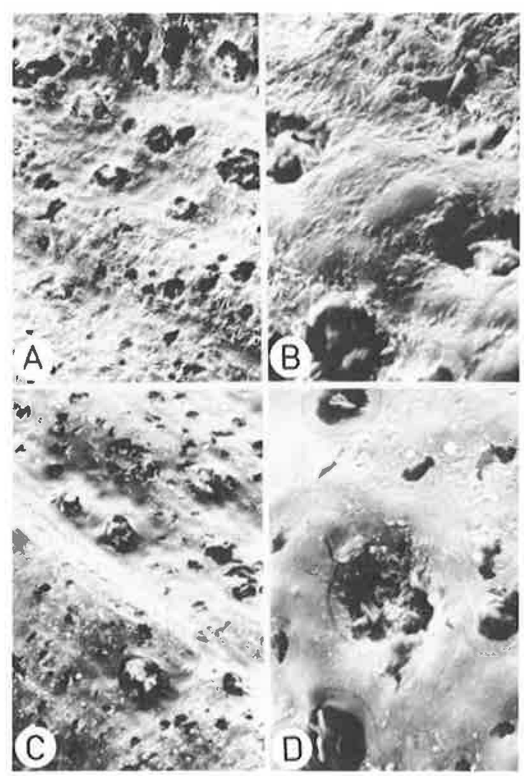



**FIGURE 4 Surfaces of salted peach twigs in late December covered with RD 2034 (a) ×1750, (b) ×5,000; or RD 2035 (c) ×1750, (d) ×5000.**

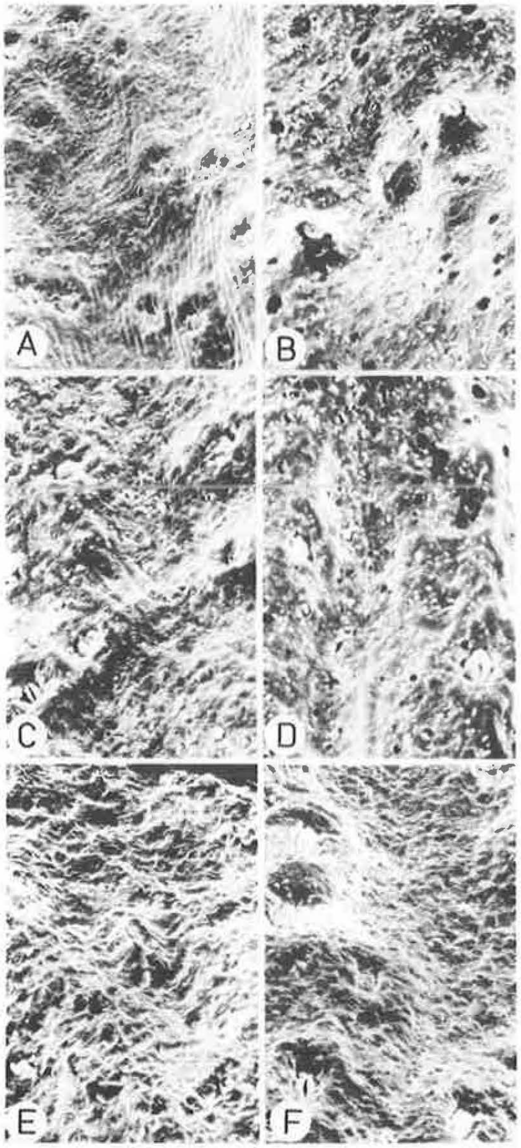



**FIGURE 5 Surfaces of peach twigs in late March after fall treatment with various film-forming products: (a) RD 1725 (×875); (b) RD 2036 (×1,750); (c) RD 2034 (×1,750); (d) RD 2035 (×1,750); (e) RD 2033 (×1,750); and (f) Folicote (×1,750).**

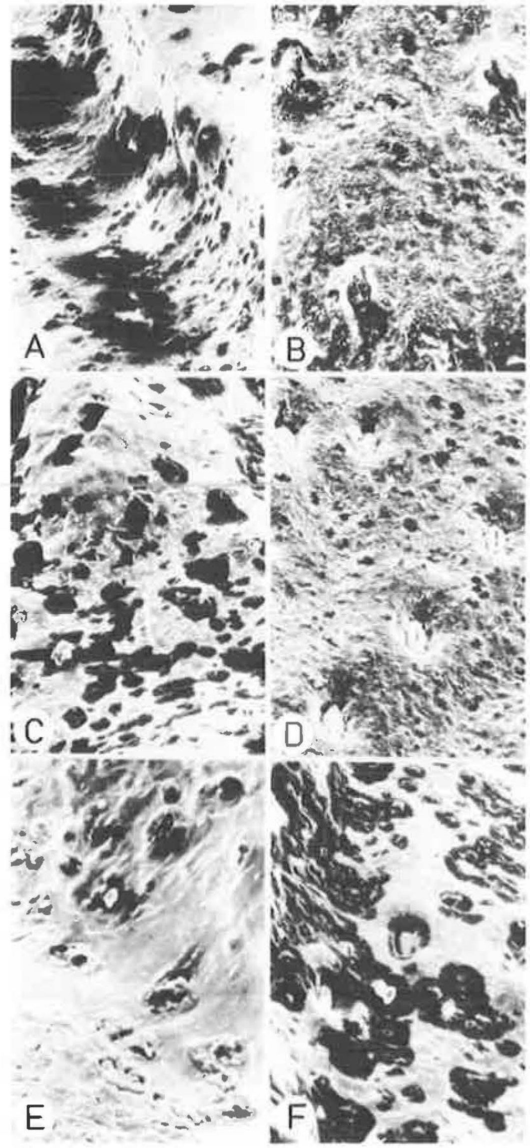



**FIGURE 6 Surfaces of peach twigs in late March after fall treatment with various film-forming products and four successive monthly applications of a 2 percent rock salt solution: (a) RD 1725 (×875); (b) RD 2036 (×2,500); (c) RD 2034 (×1,750); (d) RD 2035 (×1,250); (e) RD 2033 (×1,750); and (f) Folicote (×1,750).**

relatively mild with 1987–1988 being the most mild. Winter damage to fruit trees was below average during the winters of 1986–1987 and 1987–1988. Gale and Hagan (*7*) emphasized that field trials of film-forming antitranspirants often yield inconsistent results because varying environmental conditions can substantially alter their effects. Thus, large-scale or commercial use of any one or more of these products must await further testing under widely diverse winter conditions with other plant species.

## ACKNOWLEDGMENTS

This effort was a joint project of the Ontario Ministries of Agriculture and Food, Transportation, and Environment in consultation with the University of Guelph. Supplemental financial support was provided by International Waxes Ltd., Ontario. Special thanks to Ian Davie of that company for his many hours of technical discussion. Ted Lizak and Lou Schenck provided use of their orchards; L. W. Stobbs and J. Van Shagen assisted in the use of the SEM at Agriculture Canada, Vineland Station; and Joanne Gallagher and Bob Hamersma provided technical assistance.

## REFERENCES

1. G. Hofstra, R. Hall, and G. P. Lumis. Studies of Salt-Induced Damage to Roadside Plants in Ontario. *Journal of Arboriculture*, Vol. 5, 1979, pp. 25–31.

2. G. P. Lumis, G. Hofstra, and R. Hall. Sensitivity of Roadside Trees and Shrubs to Aerial Drift of Deicing Salt. *HortScience*, Vol. 8, 1973, pp. 475–477.
3. W. E. Barrick and H. Davidson. Deicing Salt Spray Injury in Norway Maple as Influenced by Transpiration and Humidity Treatments. *HortScience*, Vol. 15, 1980, pp. 203–205.
4. J. C. Pair and S. M. Still. Antitranspirant Effect on Leaf Water Potential and Winter Injury of Holly. *Journal of the American Society for Horticultural Science*, Vol. 107, 1982, pp. 9–13.
5. E. M. Smith, L. C. Chadwick, and K. W. Reisch. The Use of Anti-Desiccants in Reducing Transpiration and Protecting Ornamental Plants Against Winter Injury. *Proc., American Society for Horticultural Science*, Vol. 88, 1966, pp. 703–707.
6. W. C. White and C. J. Weiser. The Relation of Tissue Desiccation, Extreme Cold, and Rapid Temperature Fluctuations to Winter Injury of American Arborvitae. *Journal of the American Society for Horticultural Science*, Vol. 87, 1965, pp. 554–563.
7. J. Gale and R. M. Hagan. Plant Antitranspirants. *Annual Review of Plant Physiology*, Vol. 17, 1966, pp. 269–282.
8. K. W. Reisch. Antidesiccants Reduce Salt Damage to White Pine and Norway Spruce. *Ohio State University Experiment Station Nursery Notes*, Vol. 3, No. 6, 1970, 4 pp.
9. A. Emmons, A. Wood, and E. Sucoff. *Antidesiccant Sprays and Damage from Deicing Salts*. Minnesota Forestry Research Note, No. 258, 1976.
10. O. Piedrahita. *Prevention of Salt Spray Injury to Fruit and Ornamental Trees*. Technical Report ME-87-19. Ontario Ministry of Transportation, 1987.
11. G. Hofstra and R. Hall. Injury on Roadside Trees: Leaf Injury on Pine and White Cedar in Relation to Higher Levels of Sodium and Chloride. *Canadian Journal of Botany*, Vol. 49, 1971, pp. 613–622.
12. J. Northover. NaCl Injury to Dormant Roadside Peach Trees and its Effect on the Incidence of Infections by Leucostoma Spp. *Phytopathology*, Vol. 77, 1987, pp. 835–840.
13. M. Simini and I. A. Leone. Effect of Photoperiod, Transpiration, and Relative Humidity on Chloride Uptake of Plants Exposed to Salt Spray. *Phytopathology*, Vol. 72, No. 9, 1982, pp. 1163–1166.
14. D. C. Davenport. Antitranspirants Aid Plant Cultivation. *American Nurseryman*, Vol. 156, No. 8, 1977, pp. 28–36.


---

*Publication of this paper sponsored by Committee on Landscape and Environmental Design.*

# Wildlife Crossings of Florida I-75

GARY L. EVINK

The Florida Department of Transportation is constructing I-75 across the SR-84 (Alligator Alley) alignment in south Florida. This alignment crosses approximately 40 mi of habitat of the only known population of the endangered Florida panther. Panther movement and behavior are being monitored by radio tracking. Radio-collared panthers have been crossing the alignment, providing information on locations of crossings and habitat type being used. This information was used to develop measures to provide safe crossings of I-75 for the panther. Measures taken include creation of 23 wildlife crossings, which are 100 ft long and 8 ft high, and the extension of 13 existing bridges to provide 40 ft of land along the drainage canals under the bridges. The spacing of the crossings is roughly 1 mi apart. Fencing to cause animals to use the crossings will be 10 ft high with outriggers and three strands of barbed wire, except at crossings, where fencing will be 12 ft high with outriggers and barbed wire. Although the crossings and fencing are not completely finished, panthers and other animals are currently passing through these structures, indicating that the constructed crossings are being used.

The Florida Department of Transportation is presently constructing I-75 across south Florida. This Interstate highway is using the SR-84 (Alligator Alley) alignment in Collier and Broward counties. The limits of the Interstate project are from east of County Road 951 in Collier County to west of US-27 at Andytown in Broward County, a distance of 76.3 mi (Figure 1). This alignment severs the unique ecological areas of the Fakahatchee Strand, Big Cypress National Preserve, and the Everglades water conservation areas. Therefore, the primary concerns with the project are of an ecological nature.

Project development was initiated in the early 1980s. In October 1982, a list of potential threatened and endangered species on the project was requested from the U.S. Fish and Wildlife Service (USFWS). The USFWS reply contained six endangered species, among which was the Florida panther.

The main factors in the plight of the Florida panther are low population numbers with associated depressed genetic viability, increased human presence, diseases and parasites, and reduced prey base (*1*). The number one cause of death is highway mortality. The two major highways of concern are SR-84 (I-75) and SR-29 (Figure 1) in the area of the Fakahatchee Strand and the Big Cypress National Preserve. Eight panthers had been killed in a 13-year period on these two highways. The only known population of Florida panthers is in the area of Collier County where I-75 is planned (Figure 2). The remaining panther population is in remote areas with a population estimate of 20 to 50 animals (*2*).

During the studies for the environmental documents for the I-75 project, the possibility that the highway might jeopardize the endangered Florida panther was a principal consideration. The Interstate highway would run 40 mi across the center of the remaining habitat for the only known population of Florida panther. The panthers would need to cross the highway in order to provide food during periods of deteriorating habitat during wet seasons, to permit dispersals of young panthers, and to maintain the genetic viability of the population. Therefore, measures to provide safe crossing of I-75 for the panthers were evaluated.

Fortunately, the Florida Game and Fresh Water Fish Commission (FGFWFC) in cooperation with the U.S. Fish and Wildlife Service had ongoing studies of panther behavior and movement in the area. The Florida Department of Transportation joined in these ongoing studies to design a highway facility that would minimize impacts to the endangered panthers. The information obtained served as a basis for exploring concepts for protecting the panthers.

Although there has been a great deal of interest in highway-related animal mortality, few studies have examined causes or methods for reducing highway kills (*3*). Several possible methods of reducing highway kills were evaluated. Laterally diffusing reflectors similar to those used for deer were being tried on SR-29 and SR-84 on an experimental basis. Because it was not possible to document their effectiveness, the use of this method on the Interstate was eliminated. An increased road shoulder width for better driver and panther visibility was considered as a factor in reducing highway kills. This method was used on SR-29. However, a panther was killed on SR-29 after the shoulders were widened, so this approach was not felt to be effective. Lower speed limits are reducing panther kills both on SR-84 and SR-29 in several key locations. However, a 45-mph speed limit on approximately 40 mi of I-75 was not considered enforceable. Fencing the facility would only isolate the animals on either side of the facility. Therefore, the most viable concept seemed to be the use of wildlife crossings. Such structures had been used successfully for a variety of animals in several areas (*4–7*).

Wildlife crossings are presently being constructed on the I-75 project. Studies conducted to determine the number and location of the crossings, the crossing and fence design, and the effectiveness of the crossings for wildlife movement across the alignment are discussed.

## METHODOLOGY

As part of the studies of the behavior and movement of the panther population identified in Collier County, the FGFWFC is capturing and placing radio collars on panthers. The panthers are tracked and treed using dogs, then immobilized by drugs. Radio transmitters are placed around the panthers' necks for subsequent tracking. The animals are tracked from fixed-wing aircraft with directional antennas, as described by

Environmental Office, Florida Department of Transportation, 605 Suwannee Street, Tallahassee, Fla. 32399.

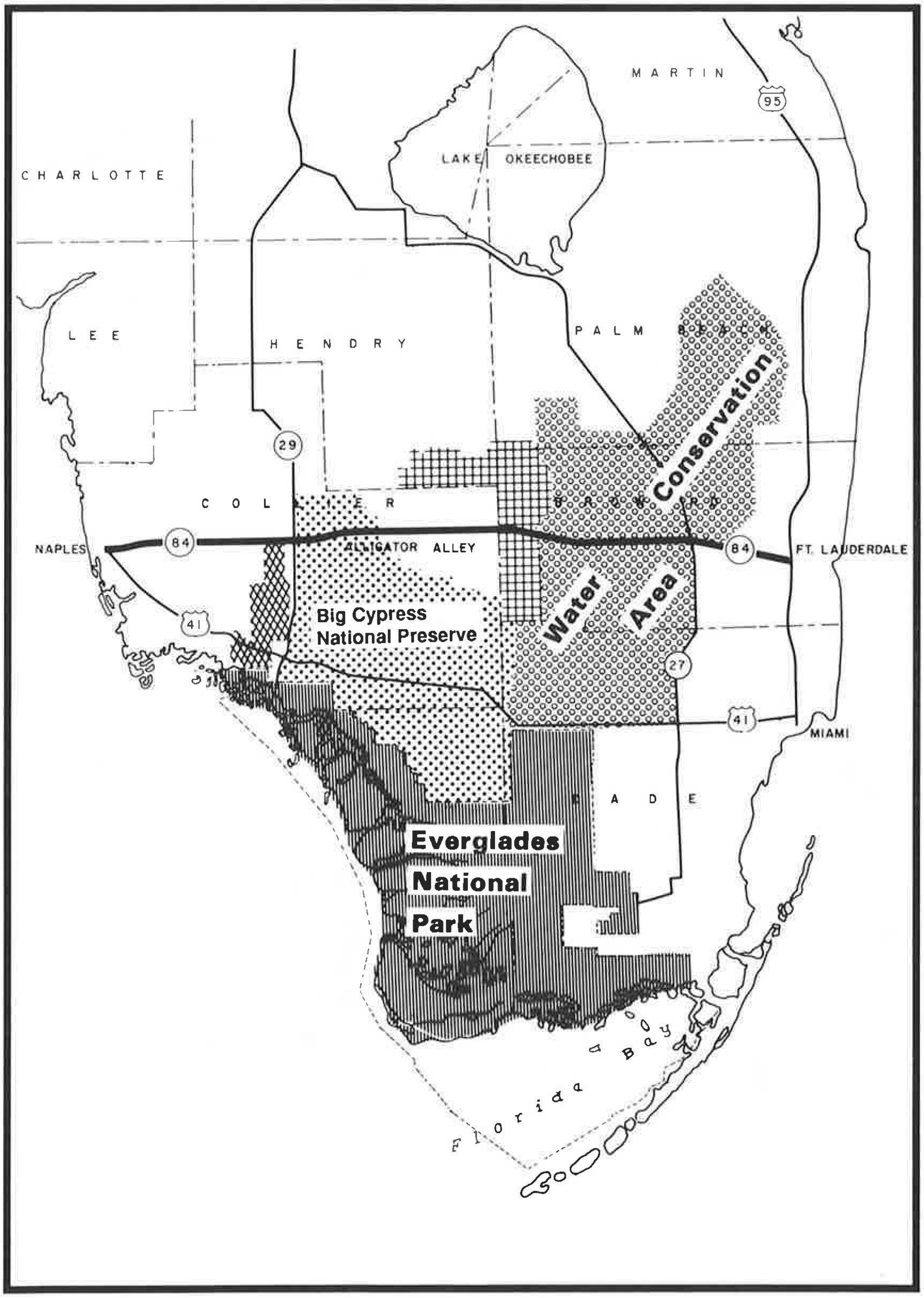



**FIGURE 1** Project and public lands location map.

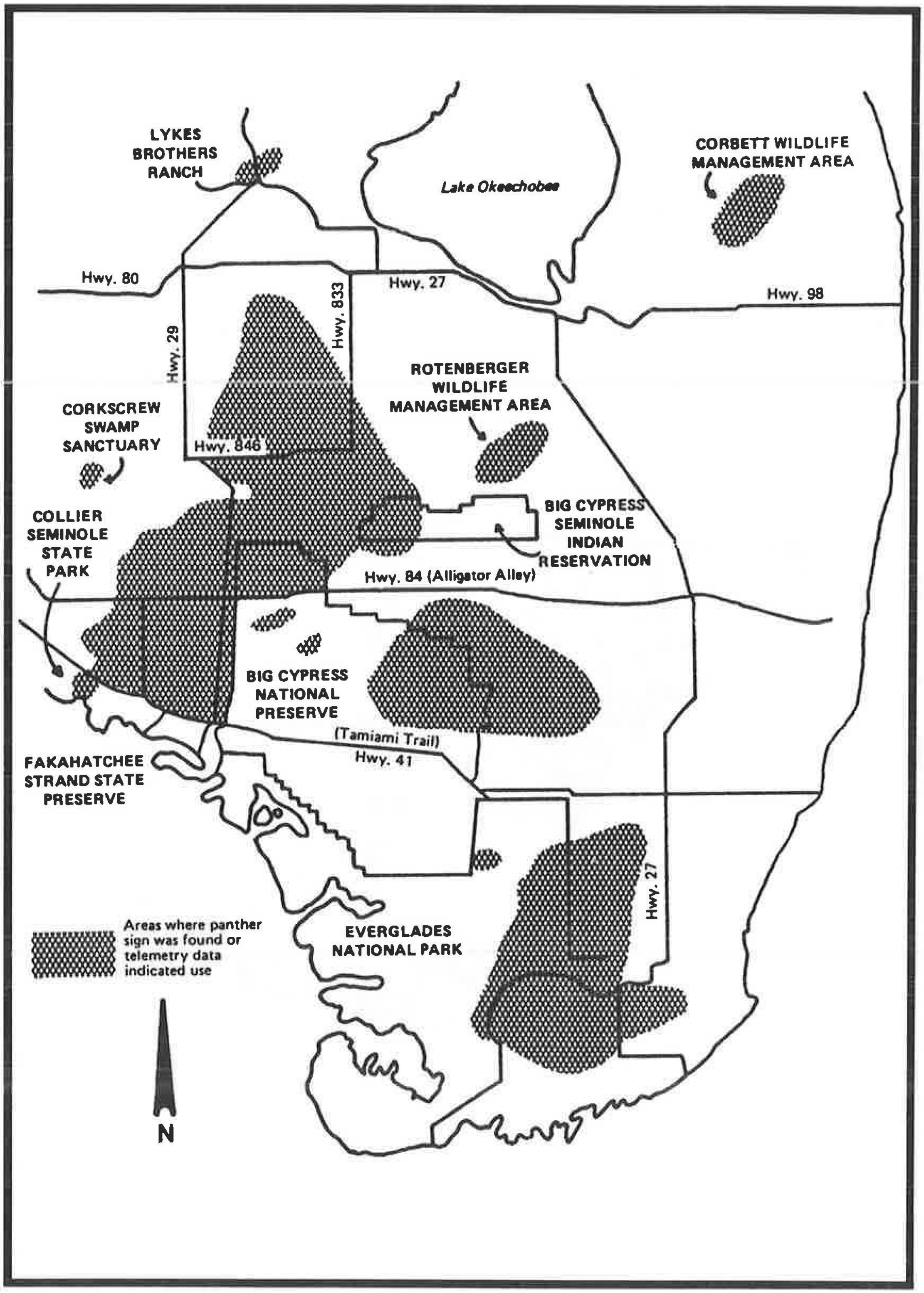



**FIGURE 2** Known range of the Florida panther.

Mech (*8*). Latitude and longitude are noted using universal transverse Mercator coordinates. Radio tracking information for 18 panthers has been used in the analysis of measures needed along I-75.

The transverse Mercator coordinates obtained from the radio tracking studies were converted to state plane coordinates, which are used by the Photogrammetry Office of the Florida Department of Transportation. The remote sensing section of this office developed land use and vegetation maps of the area using the Florida land use, cover, and forms classification system. Panther coordinates were superimposed on the land use and vegetation maps to identify the habitats being used by the panthers. Seasonal patterns of movement were mapped to depict movements during wet, dry, mating, and kitten-rearing seasons. Patterns and locations of movement across the highway were identified from this information.

Panther crossing design was based on literature review of successful animal crossings in other locations (*3–5*). The design was intended to minimize the tunnel effect of the appearance of the crossings. The idea was to present as continuous a corridor of habitat as possible with an open view of the habitat that the panther would be entering after crossing under the bridges.

Fencing height and design were determined by consulting experts with experience in captive animal facilities and panther behavior through the forum of the Florida Panther Interagency Committee (FPIC), an interagency committee of experts charged with the recovery of the Florida panther. The committee served as a forum for expert inputs on many factors related to I-75.

At present, the wildlife crossings are being randomly monitored by ground survey of footprints of panthers as described by Belden (*9*). The footprints of other animals using the crossings are also being identified. Installation of movement-activated photographic devices (Trailmaster 1500 Game Counters, Trailmaster, P.O. Box 3497, Shawnee, Kansas) is planned on completion of the crossings.

## RESULTS

Definite corridors of movement were readily identified from the radio tracking data. The shaded area shown in Figure 3 is a composite of the panther movements for five radio-collared panthers. On a macroscopic scale, it shows the areas of movement across I-75 and SR-29 for the five collared panthers.

Analysis of the locations indicated that the panthers move over large areas and stay in given areas varying lengths of time (*10*). For movement, the panthers use heavily vegetated areas such as mixed swamp forest, hardwood hammocks, slash pine-saw palmetto woodlands, and oak-pine woodlands. They often bed in saw palmetto thickets. Nighttime telemetry indicates panthers often leave dense cover and move out into open areas such as wet prairies, freshwater marsh, or agricultural land, presumably to follow the movement and feeding habits of prey (*1*).

The locations where the panthers were crossing SR-84 (I-75) were consistent with the other movement data. The panthers were using heavily vegetated corridors. Several of the crossing locations were higher ground than the surrounding areas. These were old logging tram roads, oil exploration roads, and an old railroad alignment. The points of crossing were locations where these features approached the road alignment. Another indicator that some of these locations were correct was the fact that panthers were consistently being killed by vehicles at these locations.

The vegetative mapping was then used to identify type of habitat along the remainder of the corridor in the range of the panther. From this information, it was possible to determine other probable crossing locations for areas not covered by the radio tracking studies. Several of these have since been further substantiated by the location of panther tracks in these areas.

It was felt that a number of crossings would be needed along the approximately 40-mi stretch of the known panther range. A distance between crossings of approximately 1 mi was used so that the panther and other animals would not have to travel far in finding a crossing. A combination of wildlife crossing structures and extensions of existing bridges to provide dry land crossings was determined to be the most cost-effective approach. The final design calls for 23 animal crossings and 13 bridge extensions.

The length and height of the bridges was determined by looking at various length-to-height relationships to come up with a design that did not present a tunnel appearance. Another factor was the desire to have a clear view of the habitat on the other side of the facility to present the appearance of a continuum of habitat across the facility. A length of 100 ft with a height of 8 ft was determined to be the most cost-effective design. Existing bridges will also be extended to provide 40 ft of land along the canals under the bridges (Figure 4).

In order to ensure that the panthers use the crossings rather than cross the road at the location where they approach it, fencing of suitable design was necessary. The fencing specifications call for a 10-ft-high fence with outriggers with three strands of barbed wire, except at the crossings where the fence is to be 12 ft high with outrigger and barbed wire.

Because radio-collared panthers were crossing the alignment in the Fakahatchee Strand area (Figure 1), construction of crossings in this area was begun first so that use of the crossings could be studied. The disturbed nature of the area around and under the crossings has made track identification possible. By track identification, several species of animals (Florida panther, bobcat, racoon, opossum, and armadillo) have been documented as using the constructed animal crossings. The Florida panther has gone under two of the animal crossings a few times even though the construction and fencing were not completed. The panthers move across the area at night when there are no construction activities. These are areas where radio telemetry data indicated that panthers historically crossed the corridor. Future monitoring will further document the extensiveness of use of the crossings by animals.

## CONCLUSIONS

Knowledge about the movements and behavior of the Florida panther in the project area enabled definitive development of measures to minimize impacts to this endangered species. The radiotelemetry data and behavioral information obtained on this species were critical to the decision-making process about measures to minimize impacts. State-of-the-art computer graphics that displayed land use, vegetation type, and the panther movements provided the information necessary

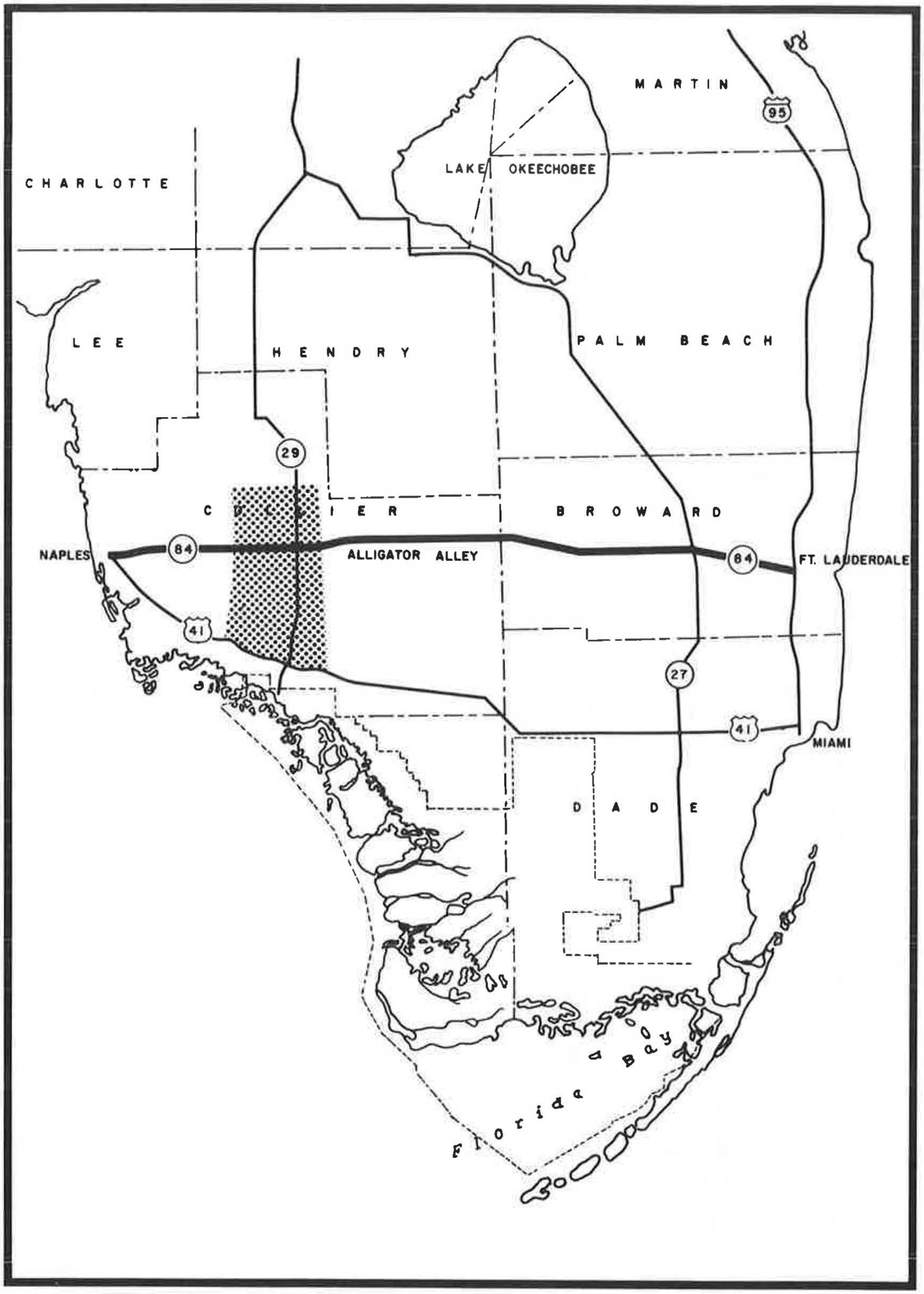



**FIGURE 3** Range of radio-collared panthers.

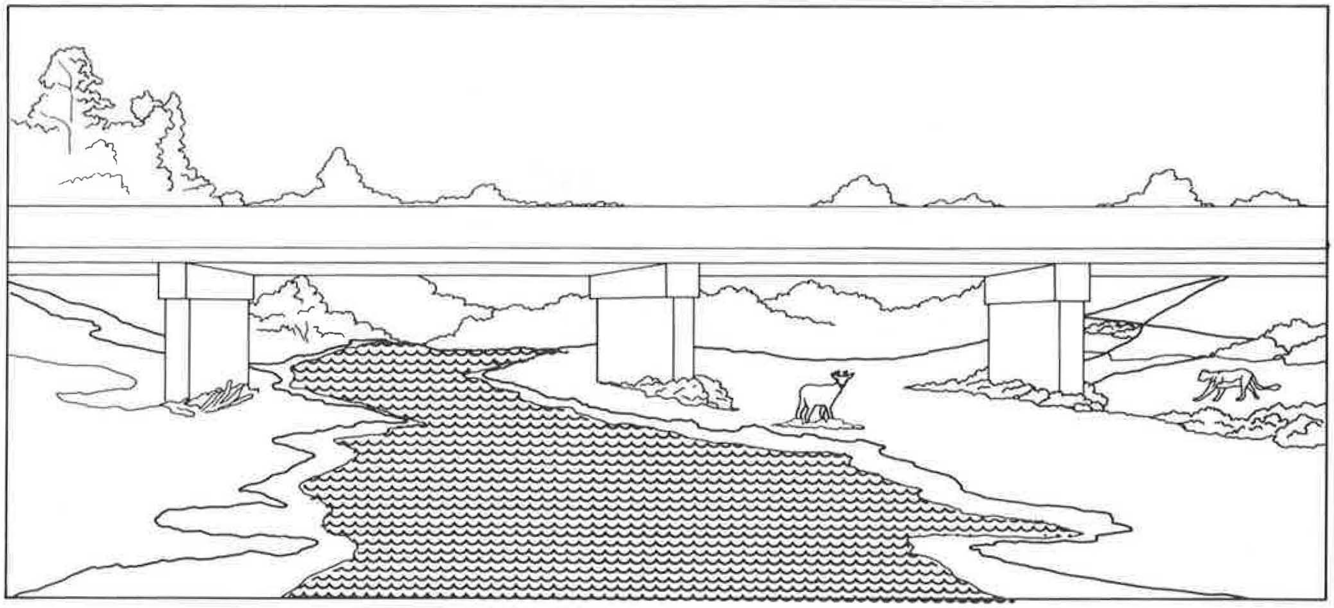


**FIGURE 4 Design of animal crossings and bridge extensions.**

to define the locations at which the panthers were crossing the highway. These techniques could be applied to many other species.

Use of animal crossing structures to provide safe passage for wildlife is proving successful. All wildlife in the project area benefit from having these safe corridors across what could have been a barrier to north-south movement in these environmentally important areas. Fencing will help ensure that wildlife does not cross the highway in areas other than the wildlife crossings.

With rapid development taking place in Florida, fragmentation of wildlife habitat has reduced the usable habitat for a number of species with large ranges. Maintaining corridors between these remnant populations is critical to the survival of these animals.

## ACKNOWLEDGMENTS

The Florida Department of Transportation appreciates the help provided by the members of the Florida Panther Interagency Committee and Technical Subcommittee as well as those persons participating in the meetings of these committees who offered their inputs on protecting Florida panthers. Appreciation is owed to Tom Logan, Chris Belden, and Dave Maehr of the Florida Game and Fresh Water Fish Commission, whose panther studies served as the foundation for decisions on the I-75 project. David Wesley, Don Palmer, and Dennis Jordan of the U.S. Fish and Wildlife Service were most helpful throughout the studies in identifying measures for minimizing impacts to the Florida panther.

Roger Whitehead, Gay Phillips, and Alan Shopmyer of the Photogrammetric Office of the Florida Department of Transportation provided the innovative computer graphic work that so clearly illustrated the panther movements and habitat that were critical to locating the crossings. Roger Eudy of the Florida Department of Transportation provided the graphics for this paper.

Portions of the research used in this study were sponsored by FHWA, U.S. Department of Transportation; U.S. Fish and Wildlife Service, U.S. Department of Interior; the Florida Game and Fresh Water Fish Commission; and the Florida Department of Transportation.

## REFERENCES

1. U.S. Fish and Wildlife Service. *Florida Panther* (Felis concolor coryi) *Recovery Plan*. Florida Panther Interagency Committee, U.S. Fish and Wildlife Service, Atlanta, Ga., 1987, p. 75.
2. D. J. Forrester, J. A. Conti, and R. C. Belden. Parasites of the Florida Panther (*Felis concolor coryi*). *Proc., Helminthological Society of Washington*, Vol. 52, No. 1, 1985, pp. 95–97.
3. D. L. Leedy. *Highway-Wildlife Relationships: A State-of-the-Art Report*, Vol. 1, Report FHWA-RD-76-4. FHWA, U.S. Department of Transportation, 1975, p. 183.
4. D. F. Reed. Mule Deer Behavior at a Highway Underpass Exit. *Journal of Wildlife Management*, Vol. 45, No. 2, 1981, pp. 542–543.
5. F. J. Singer and J. L. Doherty. Managing Mountain Goats at a Highway Crossing. *Wildlife Society Bulletin*, Vol. 13, 1985, pp. 469–477.
6. J. J. van Gelder. A Quantitative Approach to the Mortality Resulting from Traffic in a Population of *Bufo bufo* L. *Oecologia*, Vol. 13, 1973, pp. 93–95.
7. A. L. Ward, N. E. Fornwalt, S. E. Henry, and R. A. Hodorff. *Effects of Highway Operation Practices and Facilities on Elk, Mule Deer, and Pronghorn Antelope*. Report FHWA-RD-79-143. FHWA, U.S. Department of Transportation, 1980, p. 98.
8. L. D. Mech. *Handbook of Animal Radio-Tracking*. University of Minnesota Press, Minneapolis, 1983, p. 87.
9. R. C. Belden. How to Recognize Panther Tracks. *Proc., Annual Conference of the S.E. Association of Fish and Wildlife Agencies*, Vol. 32, 1978, pp. 112–115.
10. R. C. Belden. Florida Panther Recovery Plan Implementation—A 1983 Progress Report. In Cats of the World: Biology, Conservation, and Management. *Proc., 2nd International Cat Symposium*, Caesar Kleberg Wildlife Research Institute, Kingsville, Tex., and the National Wildlife Federation, Washington, D.C., 1982, pp. 159–172.

*Publication of this paper sponsored by Committee on Landscape and Environmental Design.*

# Nesting Bald Eagles (*Haliaeetus leucocephalus*) in Urban Areas of Southeast Alaska: Assessing Highway Construction and Disturbance Impacts

NATHAN P. JOHNSON

The impact of human-caused disturbance on the nesting success (productivity) of bald eagles in Southeast Alaska is discussed. The literature on disturbance of raptors generally and bald eagles specifically was reviewed. Raptor biology and behavior as they may be related to disturbance and habituation of eagles are discussed. Examples of nondisturbing highway construction completed within the standard buffer zones and time frames to protect eagles as recommended by the United States Fish and Wildlife Service for Southeast Alaska are presented. Energy (time) budget research to determine levels of (and possibly define) disturbance of bald eagles is briefly explored. A case-by-case approach to prevent disturbance of nesting bald eagles during highway construction, as required under the federal Bald Eagle Protection Act, is proposed. The approach allows incorporation of realistic, enforceable stipulations in project environmental and construction bid documents to protect nesting bald eagles, yet maximizes the flexibility necessary to schedule highway projects to minimize design and construction costs.

In Southeast Alaska (Figure 1), bald eagles (*Haliaeetus leucocephalus*) that have chosen nest sites in or near urban areas are often acclimated to high levels of human activity. The Alaska Department of Transportation and Public Facilities (ADOT&PF) has found that for these urbanized eagles, current U.S. Fish and Wildlife Service (FWS) guidelines on blasting and general highway construction to prevent disturbance of nesting bald eagles under the Bald Eagle Protection Act can be too restrictive.

The FWS basic stipulations to protect nesting bald eagles state that to permit eagles to initiate nesting activities there should be no heavy construction work within 100 m of a nest from March 1 to May 15, and this period should continue to August 31 if the nest is occupied (*1*). If the nest is not occupied by May 15, construction activities within 100 m can proceed. For blasting, the timing restrictions remain the same, but the buffer zone is 800 m.

Some recent ADOT&PF projects have involved blasting and heavy equipment work near eagle nests within the FWS buffer zones and time frame. The pairs of eagles using these nests successfully raised young during the affected nesting seasons. In addition to this field information, ADOT&PF undertook this study to evaluate the existing literature on disturbance of nesting eagles and methods of monitoring disturbance. On the basis of the findings of the study, the department recommends that the FHWA develop a memorandum of agreement (MOA) with the FWS to (a) on a case-by-case basis, mitigate or monitor potential impacts from construction on eagle nest trees to prevent disturbance, and (b) undertake research to better define disturbance.

Increases in location and design costs caused by mitigation or monitoring on a case-by-case basis will be more than offset by the minimization of both construction delays and elevated costs caused by the presence of active eagle nests adjacent to highway construction projects.

## BACKGROUND AND HISTORY OF DISTURBANCE STUDIES

The federal Bald Eagle Protection Act of 1940 prohibits the taking of bald eagles (including nests or eggs) at any time or in any manner without a permit. As defined in the Act, "taking" includes "molest or disturb"; however, nowhere in the Act (or implementing regulations) are these two terms defined. To date, case law offers the only definition of what may constitute "molest or disturb."

The eastern region of the U.S. Forest Service implemented a policy of establishing buffer zones around individual bald eagle nest trees in 1963 (*2*). Whether the FWS concept of buffer zones evolved from this policy or was established independently is unclear.

### Early Studies

Early investigations of potential impacts of human activities on nesting bald eagles have been documented in the literature (*3*,*4*). Quantification of impacts in these studies has been general, focusing on the human activities involved, then attempting to measure nest abandonment or lowered productivity as an indication of disturbance. Nests were usually grouped into disturbed and undisturbed categories.

One of the first studies to evaluate human disturbance as a potential cause of nesting failure among bald eagles was carried out in the Chippewa National Forest in Minnesota (*2*). Results indicated specific types of human activities did not significantly disturb nesting eagles. A major component

Alaska Department of Transportation and Public Facilities, P.O. Box 021467, Juneau, Alaska 99802–1467.


**FIGURE 1 Alaska and the Lower 48.**

of the disturbances was human recreational activities that took place from mid-June throughout the rest of the summer. These activities occurred after nests were established and the young hatched, the two most critical time periods from a disturbance standpoint. Nest occupancy and fledging of young were used as measures of nesting success.

Two other researchers (*5*) on the Chippewa National Forest classified four different levels of disturbance within 1 mi of nests. Analysis of the data showed a negative relationship between both apparent nesting activity and measured production as compared to degree of disturbance. The ratio of activity to productivity was better with lesser disturbance.

A study on the Kenai National Wildlife Refuge in Alaska (*6*) separated eagle nests into disturbed and undisturbed categories. Human disturbance was not quantified, and apparently no statistical analyses were made of the productivity data, but the study indicated human disturbance can decrease productivity.

Another study in the Chippewa National Forest (*7*) found no evidence that under management policies at that time, natural or induced human activities had any major impact on bald eagle reproductive success. The authors concluded that, "birds at unsuccessful nests, as a group, were not exposed to higher levels of human activities than birds at successful nests." The investigators went so far as to suggest:

> [E]xperiments in which a substantial number of eagles are disrupted to the point of nest failure by a variety of human activities will have to be carried out in a number of different areas in order to address this question (of the affects of human disturbance on nesting eagles) adequately. The relatively stable population of eagles in Alaska and Canada could be used in such studies.

The use of the word "disrupt," i.e., to break apart, rupture, to throw into disorder, or to cause to break down, implies that levels of disturbance that do not cause nest abandonment are acceptable. This approach is extreme and unnecessary. The investigator's straightline approach toward a nest, with pausing at 20-m intervals in plain view, until the attending adults flush, is unnatural human behavior and is directly threatening to nesting birds. The technique may have been designed for statistical analysis rather than duplicating normal human-induced disturbance factors. The principal investigator of the FWS Eagle Management Studies Program in Southeast Alaska (Jacobson, unpublished data) agrees that any direct threats by man can significantly impact breeding behavior and success.

More recent work in western Oregon (*8*) characterized 201 bald eagle nest sites in three different forest types over four nesting seasons. Mean productivity was "lower at sites altered by logging or other human disturbance," particularly clear cuts, main logging roads, and nonrecreational human activities. In given nesting territories, most newer, more recently used nests were farther from human activities than associated older nests in these same territories.

The researchers measured many variables to characterize individual nest trees, the forest stand surrounding each nest tree, and human activity. Many of the human activity categories were actually measurements of habitat alteration over time rather than direct impacts of day-to-day human activities on nesting birds. Clearcut logging and associated roadways plus nonlogging roads and highways, public facilities, and private homes were some of the major human activities measured.

Other studies (*9–11*) have also demonstrated lowered productivity and site desertion associated with human disturbance at bald eagle nest sites.

### Activity Budget Approach

The current approach to quantifying impacts of disturbance to raptors is typified by the use of the activity or energy budget on peregrines in the Sagavanirktok River drainage in Alaska (*12*). The technique consists of determining the energy budgets of undisturbed nesting birds and then statistically comparing them with the energy budgets of those same birds (or other nesting pairs) under disturbed conditions. The energy budget is the amount or percent of time (or energy) expended by an animal in performing various behavioral activities as determined through field monitoring. No significant difference in the two sets of data indicates no impacts to breeding success from the disturbance under consideration.

In this study, behavioral and environmental data were recorded both on activity and disturbance forms for each half-hour of observation at each nest site. Observations focused on the attending adult at the nest or in the adjacent cliff area. During experimental disturbances, intensive observations were made on the focal bird. Each recorded disturbance was described by several characteristics: (a) behavior of the birds before disturbance, (b) type of disturbance (other species, helicopter, light truck), (c) degree of reaction of the birds (none, mild, moderate, severe), (d) duration of disturbance (time within restricted zone), (e) duration of reaction of the birds, (f) direction (in relation to falcons), (g) distance (closest linear distance to falcons for all disturbances, and altitude for avian predators and aircraft), (h) noise level (none, low, medium, high), and (i) visual stimulus (none, unlikely, probably, positive).

Experimental disturbances included construction and maintenance equipment, airplanes, river boats, snow machines, and people on foot. The type and timing of experimental disturbances were varied to simulate both normal and unusual disturbance activities.

The author tested the "hypothesis that time spent in each activity category did not differ among the two disturbed and the undisturbed activity budgets. . . ." He then used a battery of nonparametric analyses of variance to determine levels of significance. He concluded that the disturbances studied "did not cause significant changes in the time spent in important behaviors (e.g., incubation), and did not cause measurable impacts on occupancy or productivity."

Although no significant differences in activity budgets with regard to specific human activities indicates no disturbance, significant differences may begin to define disturbance from a biological standpoint, that is, by causing reduction in current and future productivity. For example, operation of heavy equipment adjacent to a nest in the early morning hours may significantly reduce parental feeding behavior of newly hatched young to the point of lowering productivity.

The basic activity budget approach is also applicable to bald eagles (*13*). This pioneering study on quantifying the nesting activity (time) budgets of bald eagles in Southeast Alaska concluded, "Detailed accounts of nesting time budgets are needed to develop criteria for bald eagle management in areas where the potential for human disturbance is of concern." Remote, time-lapse movie cameras were used to "document the amount of time adults spent at incubating, brooding, and feeding at the nest, with specific emphasis on: the division of these activities between the male and female, temporal changes in time budgets, and the effects of several environmental parameters on nesting time budgets."

Time-lapse photography provided instantaneous samples, single-frame exposures every 90 sec. The films were developed and then analyzed with a time-lapse analyzing projector. Activity data were punched directly into a computer for analysis. Results indicated significant differences both in individual and pair activity budgets with regard to human disturbance, incubation, brooding, prey deliveries, feedings, and effects of weather on nesting activities. With respect to disturbance, reactions were variable, "but . . . most eagles were extremely sensitive to intrusion during incubation and for the first one or two weeks after hatching."

Video equipment has also been used to monitor nesting bald eagles in California and Arizona (*14*). These continuous real time observations lend themselves to a variety of analyses unlike time-lapse photography, which records data at preset intervals.

### Habitat Disturbance

Observations and data collected during most of the disturbance studies attempted to measure only the direct impacts of human activities on the nesting eagles themselves. The more important long-term problem of the loss of bald eagle nesting habitat caused by human activity (disturbance) must always be kept in mind (*1,7,8,15*). Existing nest trees will eventually be lost from one cause or another, such as decay, blowdown, or human activities, and, therefore, over the long term, alternative sites must be available to maintain viable eagle populations.

In a study of the relationships of bald eagle nesting to forestry practices near Petersburg, Alaska, from 1967 through 1969 (*15*), nest sites located in the fringe of timber left along the beach as a result of logging were found to be highly susceptible to wind throw. In one winter, 1968–1969, 20 percent of the known nests were lost to storm damage. Buffer zones of 660 ft and reduction of beach strip logging to ensure potential nest sites were recommended.

In an attempt to minimize impacts on eagle nests and nesting on federal lands in Southeast Alaska, the U.S. Forest Service (FS) and the FWS entered into a memorandum of understanding (MOU) in 1968. It requires the FS to "establish and maintain a minimum five-chain radius habitat management buffer zone around each bald eagle nest tree and exclude all land use activity within the zone." It also provides a mechanism for possible variances to these buffer zones. However, the FS and the FWS jointly agree that to "maintain the bald eagle nesting population at natural levels of abundance, a sufficient number of trees, suitable for supporting eagle nests and properly distributed along the shoreline, must be present in perpetuity." Neither "natural levels" nor "sufficient numbers" are defined.

In 1979 and 1980, bald eagle nests in Southeast Alaska were surveyed before and after logging to assess the adequacy of the 100-m buffer zone to protect nests and nesting habitat (*1*). Few of the clearcuts in the study were adjacent to the 100-m buffer zones. However, had clearcuts been adjacent to all buffer zones, "loss (due to wind throw) would have averaged 17 percent of the buffer zone after just a five-year period." If the clearcuts had "surrounded the 100-meter buffer zone, potential would exist for much greater losses to blowdown." The author concluded, "the loss of nesting habitat from blowdowns adjacent clearcut areas will probably cause the most serious long-term problems for eagles under the existing management policy." Similar problems have been documented in the coterminus states (*8*).

The potential loss of future nesting habitat becomes heightened in urban areas where land ownership shifts from unreserved public lands (those left in their natural state) to public use and private lands. The Bald Eagle Protection Act can be implemented to protect nesting eagles and existing nest trees but cannot exclude construction of highways, homes, businesses, and other urban amenities in areas that may some day provide future eagle tree nest sites. Thus, the availability of potential nest trees may depend on reserving parcels of unreserved public lands and fortuitous retention of suitable sites on private lands.

### Legal Definition of Disturbance

Even though evidence clearly demonstrates eagles can be disturbed to the point of deserting their nests and young, legal action to halt such activities seems to require proof of negligence or show of intent to do harm. The Bald Eagle Protection Act itself states, "Whoever . . . shall knowingly, or with wanton disregard for the consequences of his act take . . ." A case in point (*16*) involved an eagle nest in the Juneau area on private property that was being developed. The owner was observed clearing and burning brush near the nest site in March. He was informed of the presence of the nest, given a copy of the Bald Eagle Protection Act, and advised not to disturb the birds from March through July. He indicated he would not disturb the area. The eagles selected the site and nested. In late April, the owner, disregarding his earlier statement, began clearing and burning again. Drifting smoke disturbed the adult eagles. The owner was warned again. The adults abandoned the nest in late June. Subsequent field investigation revealed a dead eaglet at the base of the nest tree. The owner and an employee were each fined $200 for what Schempf called an "open and shut case of willful disturbance that ultimately caused the death of the eaglet."

Although existing case law may define disturbance from a legal standpoint, there is a difference between the point of successful criminal prosecution and a more conservative point of acceptable management impacts associated with disturbance (Schempf, unpublished data). From both legal and biological standpoints, there is a need for a functional, biological definition of disturbance. An emerging approach for defining disturbance is maintaining long-term productivity. Assuming adequate food resources, the number of available nest sites and the number of young raised per nest site each year are the key factors of the long-term productivity equation. Of course, productivity data must be balanced against mortality and survival rates.

Questions that must be addressed in fine tuning this definition are as follows:

- Should the definition include an assessment of current and potential levels of bald eagle productivity?
- Should it include measurement of lowered productivity during the time of disturbance?
- How would productivity be measured?
- Would the definition require abandonment of nest, eggs, or young?
- Would successful nesting in successive years counterbalance specific levels of disturbance from human activities, particularly during years of high eagle populations?

Development of a functional definition of disturbance is also in the best interest of state and federal highway agencies. It should lead to more cost-effective and expeditious development of public works projects.

## RAPTOR BIOLOGY AND BEHAVIOR, EFFECTS ON POTENTIAL NESTING DISTURBANCE

For most raptors, the main habitat requirements for nest selection and successful rearing of young are (a) adequate food supplies before and throughout the breeding season, (b) a satisfactory nest site with associated perching areas, and (c) visibility of adjacent territory or feeding grounds (*4,17–20*). The more completely these three conditions are met, the less raptors are disturbed by human activities.

Work with peregrine falcons (*Falco peregrinus anatum*) in the Yukon Territory (*18*) indicates that "physiological condition of breeding females may be the key factor in regulating annual breeding success." Breeding success was considered to be strongly and inversely tied to the energy requirements expended during spring migration by breeding females and could affect the psychological as well as physiological conditioning of the birds. Does this type of preconditioning also affect the breeding success of bald eagles? Evaluating the physiological condition of nesting eagles may be a base ingredient in any monitoring program and should include quantitative and qualitative measures of available food sources within individual nesting territories.

An interesting situation with respect to preconditioning in nesting bald eagles seems to occur annually in the Chilkat Valley near Haines, Alaska (Jacobson, unpublished data). Observations during late spring nesting surveys conducted by the FWS in the middle to upper Chilkat Valley show average, though often variable, densities of active nests. However, their observations during production surveys flown later in the summer indicate very low nesting success. The middle Chilkat Valley, with its abundant winter food source of spawning salmon, is an important over-wintering area for bald eagles, particularly young birds. A certain percentage of young and maturing birds may orient to the area, making their first nesting attempts there. During the spring and early summer, the large spawning runs of salmon are not present, however. The low nest success rates may be from inexperience, or the combined impacts of high nesting density and inadequate food supply. These nesting pairs may be severely stressed, making them susceptible to even low levels of human disturbance. In this situation, any loss of productivity caused by human disturbance of a marginal breeding population may be insignificant. Also, early termination of what would normally be an unsuccessful nest may free up food resources for another marginal pair to raise their young to fledging.

Human activity may also increase the local food supply and thus concentrate eagles (*21*). Bald eagles frequently used a garbage dump on Amchitka Island, Alaska (*22*). A high percentage of use was by subadult eagles; however, adults did use the dump as a supplemental food source. During the winter and early spring months, the dump may have been an important supplemental food source for young birds and potential nesting pairs.

An experimental winter feeding program for eagles was carried out in Maine from 1981 through 1985 (*23*). During this period, 98,000 kg of carrion were dispensed at feeding stations in four major eagle wintering areas. First- and second-year birds became heavily dependent on the artificial food source, with older birds less dependent. Analysis of banded birds showed productivity of local populations near feeding sites was enhanced.

The relative health of any population under study must be considered along with preconditioning when attempting to determine the effects of human activities on nesting bald eagles. The estimated bald eagle population of Southeast Alaska was approximately 7,000 adults for both 1967 and 1977 (*24*). In the FWS's Seymour Canal Study Area in Southeast Alaska, productivity exhibited a broad scale decline in 1979, 1980, and 1981, dropping by almost 50 percent for unknown reasons (*1*). The most recent aerial census of bald eagles in Southeast Alaska indicated a total adult population of close to 12,000 birds (Jacobson, unpublished data).

The bald eagle population may be peaking in Southeast Alaska (Jacobson, unpublished data). The rate of population increase is slowing and reproductive rates are dropping off.

With large population fluctuations over an extensive area in Southeast Alaska, there remains a provocative question that should be addressed in any definition of disturbance from a biological standpoint. What is the real biological impact of 1 year of reduced or missing production from one to a few nests either on a local population or the larger regional population? Long-term cumulative impacts of individual projects must also be considered.

Another important variable that must be considered is the individuality of the birds. For peregrine falcons (*25*), variations "in response to a disturbance exists between individuals, . . . in one individual over time, . . . and in one individual's reaction to different types of disturbance." Also, "a complex array of factors may influence a peregrine's response to disturbance, and perhaps more important, the reaction of the falcon in any particular instance is highly unpredictable." Factors that may affect a given bird's response to disturbance are "nature of the disturbance, type and severity, frequency and duration, distance from nest site, height of nest above river, presence of intervening topographical features, time relative to reproductive phenology" and "sex, age and breeding status of the individual(s)."

This same difficulty of predicting the effects of a given type of disturbance applies to individual bald eagles because of their variable responses to human activity (*19*).

The variability of reactions of individual bald eagles to the climbing and placing of cameras in eagle nest trees or adjacent trees was documented in a study of bald eagle nesting activities on Admiralty Island, Alaska (*13*). One female returned to the nest while the camera was still being mounted in a tree less than 30 m from the nest. At another nest, the female returned within a few minutes of the researcher's descent from the camera tree. At a third site, the male was the first to return, but not until nearly 2½ hr following camera installation.

The individuality of raptors also influences the degree to which particular birds or pairs of birds can become habituated to human activities (*7,26*). Habituation is the nonreaction of an animal to nonthreatening, usually repetitive events, although there is often a behavior threshold beyond which the involved disturbance is unacceptable. At that point, avoidance behavior sets in and nest abandonment may occur. This threshold, for raptors in general (*12*), is "influenced by season, age, sex, previous breeding experience, health of birds, weather and/or prey availability." In the Admiralty Island study (*13*), some eagles abandoned nests because of installation of nest monitoring cameras. However, in the 2nd year of the study, the researcher made regular visits to the study nests before installation of the cameras and found it greatly reduced nest abandonment.

Analysis of data gathered on the Chippewa National Forest (*7*) suggests "eagles avoid human settlements when building new nests." Settlements consisted of clusters of houses occupied throughout the year. The availability of nest trees in the area was not the limiting factor (Mathisen, unpublished data). However, on the basis of recent observations, some newer territories (1986–1988) have been established closer to the housing areas. This proximity is probably a result of habituation and the population's approaching saturation density. Current nesting data indicate a slowing of the population growth rate coupled with a reduction in productivity.

The fact that bald eagles nest and successfully raise young in urban areas demonstrates that the required nesting habitat is present and any needed physiological preconditioning dependent on availability of foods has been met. Man-caused disturbance factors are usually greater in urban than wilderness or rural areas, so it follows that these breeding pairs of eagles are tolerant of, or have become habituated to, some degree of human disturbance. Several current researchers (Ambrose, Cain, Lincer, Mathisen, and Ritchie, unpublished data) agree.

From 1981 through 1987, 215 nestling bald eagles have been captured by the FWS in Southeast Alaska for translocation to the contiguous 48 states (*27*). Most of these birds, 180, came from the Chatham Strait study area, which mainly includes the eastern coastline of both the lower Chilkat Peninsula and Chichagof Island. These 180 eaglets constitute a 59 percent removal of the 303 young available on the entire study area over the 7-year period. A control area is located near the removal area. Study data show "an increasing trend in production of (total young) for the experimental area and a decreasing trend for the control area." The high productivity rate could be caused by the removal of the nestlings, which "may have actually created a positive reproductive response in the experimental area." In addition, the number of young raised per occupied nest was identical for the experimental and control areas. Therefore, the author concludes, "no detrimental effect on productivity has been detected from removal of young during the 7-year study period."

Recent work by the FWS Eagle Management Program indicates nest densities along the Juneau road system, particularly the Auke Bay area, are higher than in many nonroaded portions of Southeast Alaska. Also, productivity appears to be comparable to, or in some cases exceed, other surveyed areas.

The FWS has collected several years of nesting success data both for the Juneau urban area and the remote Seymour Canal study area on Admiralty Island. These data should be analyzed to determine the degree to which overall impacts of urbanization have affected long-term eagle nesting success and productivity.

The argument can be made that the Mendenhall River estuary, biologically rich Auke Bay marine waters, and associated uplands are prime eagle nesting habitat and that eagle nesting densities and productivity were substantially higher before urbanization. Although this may be so, unfortunately, no historical productivity data are available to substantiate this hypothesis. On the other hand, the data indicate that as long as nest sites are available, the eagles will occupy them and successfully produce young at rates similar to nests in nonurbanized areas. This would tend to indicate the limiting factor is the number of available nest sites (or territoriality) rather than food supplies or disturbance by human activities in the area. The head of FWS Southeast Alaska Eagle Management Studies feels there is no one limiting factor (Jacobson, unpublished data). He feels food supplies may be the key. If food is plentiful and trees are available for nesting, then the eagles will use the trees to nest.

The bald eagle population may be peaking in Southeast Alaska (Cain, unpublished data; Jacobson, unpublished data). The rate of population increase is slowing and reproductive rates are dropping off.

## URBAN EAGLES IN SOUTHEAST ALASKA—THE NEED FOR CASE-BY-CASE ASSESSMENT

As demonstrated in the four cases to be discussed, the bald eagle's tolerance of, or acclimation to, human disturbance in urban areas, at least in Southeast Alaska, can be high (Figure 2).

### Stabler Point

This nest is located along biologically rich Auke Bay, near Juneau, Alaska. Before highway construction in the area, the nest tree was approximately 50 m from the edge of a 20-m rock cliff. Following highway construction, the nest was less than 15 m from the edge of the cliff. Historically, the nest has been regularly productive. Eagles successfully raised two young in the nest during the 1981 and 1982 nesting seasons when removal of the rock face through the use of explosives and general highway construction activities occurred.

Recommendations in the ADOT&PF construction contract required blasting within 800 m and general construction activities within 100 m be suspended during the March 1 to April 30 nest selection period. If the eagles selected the nest, the restrictions would continue through August 31. If they did not select the nest by April 30, construction could resume.

In 1981, the contractor did not finish drilling and blasting by March 1 and asked for a 3-week extension. The FWS required blasting and construction to be monitored to prevent substantial disturbance of the nesting eagles. Nine rock blasts were monitored from March 3 through March 13, 1981. During these shots, eagles attending the nest flew nine times (64 percent) and did not fly five times (36 percent). Other reactions such as raising wings and staring in the direction of the blasts indicated some level of disturbance.

Construction noise levels measured at the base of the nest tree ranged from 40 to 50 dBA. Light planes flying nearby registered 55 to 65 dBA. Background noise levels ranged from 40 to 50 dBA. Aircraft overflights were in the mid-50- to 70-dBA range with peaks at 75 and 80 dBA.

At least seven shots occurred the next year from March 2 through March 17. Reactions of the nesting eagles were not monitored, nor were any noise measurements taken.

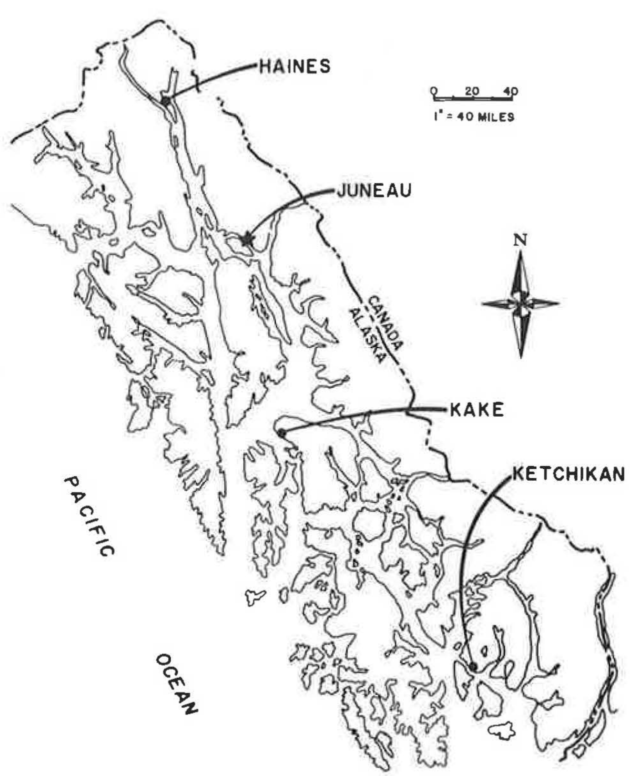



**FIGURE 2 Southeast Alaska.**

### North Tongass

This project consisted of reconstructing the North Tongass Highway from the Ketchikan city limits to the Ward Cove bridge. One large area of rock blasting occurred in the Ward Cove cannery area. Two eagle nests are located near the rock removal area, one at about 230 m distance and the other at about 500 m. Over the past several years, one or the other nest has been occupied; however, during the 1988 blasting period, both nests were occupied (Jacobson, unpublished data). At least one young was fledged at each site.

In March of 1988, ambient noise levels, mainly caused by aircraft traffic, were measured twice at the nest nearest the blast area. Noise levels from 18 aircraft were measured during 1 hr on the 1st day and from eight aircraft during 1 hr on the 2nd day. Noise levels generated by these aircraft generally were in the mid-50- to mid-60-dBA range. Two helicopter flyovers registered 65 to 67 and 75 to 76 dBA. The loudest noise levels were produced by two Dehavilland Beaver aircraft, 78 and 94 dBA. General highway traffic noise averaged in the 40-to-50-dBA range with highs in the 50-to-60-dBA range. Two rock blasts were monitored at a point 60 m closer to the blast from the nest site. One registered 54 dBA; the other less than 50 dBA.

At the nest farther from the blast area, ambient noise levels were monitored for only one 1-hr period and no blasts were monitored. Again, aircraft were the main generators of noise, with 10 overflights. Half of the aircraft registered in the 50-to-60-dBA range. Two helicopters measured 63 to 66 dBA, two Beavers registered 60 to 67 dBA, and one unknown aircraft registered 70 to 72 dBA. General highway traffic noise ranged from 40 to 50 dBA. This site was noticeably quieter.

Both nests are well within the 800-m buffer zone for blasting recommended by the FWS. Blasting and removal of the first lift of rock occurred before eagle nest selection. Succeeding blasts were below the edge of the cliff, which was oriented away from the eagle nests. The blasts were small, generated velocities of less than 2 ft/sec at 30 m distance, and occurred on a regular basis, usually at 10:00 a.m. daily.

### Fred Meyers

This nest is located to the north behind Fred Meyers. It is 15 to 20 m from the Old Glacier Highway in Juneau and has been used regularly for a number of years.

In 1988, firewood logging occurred throughout the nest selection period. Some trees within 10 m were felled. General noise levels at the base of the nest tree were monitored in mid-June during a 1-hr period from 3:00 to 4:00 p.m. General highway noise from the Egan Expressway (approximately 400 m distant) ranged from the mid-50s to the low-60s dBA. Peak vehicle noises and light planes at the Juneau International Airport averaged 68 dBA. Nineteen sight-seeing helicopter overflights averaged 78 dBA. The helicopter flights most likely started in mid-May with the beginning of the tour boat season. This disturbance would have followed nest selection and probably hatching. Also in mid-June, a bulldozer was used to grade the vacant lot across the Glacier Highway at about 75 to 100 m from the nest. FWS personnel on a helicopter survey, July 27, 1988, found two young in the nest. On August 31, 1988, one fledged young was seen perched near the nest.

### Kake

This nest is located adjacent to Keku Road about 1.5 km south of Kake and just north of the Alaska Marine Ferry Terminal. No noise or other disturbance data are available for this nest, which has been regularly active over the last several years. The nest tree is located approximately 30 m from the centerline of Keku Road, 30 m from the communities' diesel-fueled power generating and transformer station, 40 m from an active fuel tank farm, 10 m from fuel supply lines, 60 m from a service station, 70 m from a heavy equipment maintenance station, 75 m from a new port facility, 45 m from an operating cannery, and 170 to 330 m from an intermittently used rock quarry. All of these facilities are in plain view of the eagle nest. Also, heavy equipment from road graders to logging trucks frequently traverse the road.

The conclusions of the following study probably apply to all raptors, including the bald eagle and the mandates of the Bald Eagle Act. The work deals with a study of the protection of peregrine falcons from disturbance under the Endangered Species Act of 1973 (*25*) on the basis of a review of the literature and the results of a questionnaire the FWS sent out to biologists who have worked closely with the peregrine and other raptors in Alaska.

Citing several cases documenting the variability of reactions among individual peregrines to human disturbance, the researcher concludes, "it is extremely difficult to draw upon observations of individual birds or pairs to make inferences about the sensitivity or behavior of an entire population or species." This same variability of peregrines to a particular response "poses something of a dilemma to (any) attempt to develop protection measures."

The author acknowledges that the current recommended restrictions on human activities near peregrine aeries "are not inviolable." They are intended to aid responsible agencies as to whether proposed activities may affect the peregrine. When a proposed action might violate any of the restrictions, the initiator of the action "must enter into consultation with (the FWS) to examine in detail the proposed activity and its effect on" the peregrine. This type of "biological assessment" is required under Section 7 of the Endangered Species Act.

Two pertinent responses quoted from the review of the questionnaire are as follows:

1. "All respondents affirmed that the distance at which restrictions should apply should depend on the nature of the activity, time during the breeding season, and local topography. The desirability of a case-by-case review was expressed."
2. "All respondents agreed that human activities should be restricted near nest sites. Approximately 50 percent of (the) biologists who answered the question qualified their answers, stating that the nature of the intrusion, distance from eyrie, and presence of intervening topography should be considered, and that human activity need not be restricted in all cases."

Researchers in Minnesota (*7*) concluded, "Not only are individual eagles likely to differ in their response to disturbance, but the same eagles may respond differently at different times. . . ." Because this tolerance to human disturbance can vary among populations, they strongly recommended that "buffer zones be based on data from each managed population and, to the extent possible, from observations of specific pairs of eagles." This recommendation supports the concept for creating management plans for individual pairs of nesting eagles (*28*). Several other researchers agreed that guidelines need to be developed on a case-by-case basis (Ambrose, Grubb, Schempf, and Ritchie, unpublished data).

The general application of the FWS guidelines (800 m for blasting and 100 m for general construction during nest selection and nesting) in urban areas certainly may not always be appropriate. Case-by-case analysis in FHWA project development procedures should expedite needed public works projects and save money, yet adequately maintain nesting viability of bald eagles in urban areas. However, case-by-case analyses will have to be based on field research, particularly activity (time) budget studies tailored to specific bald eagle nesting situations.

## RECOMMENDED APPROACH

The following procedure for assessment of potential disturbance of nesting bald eagles on a case-by-case basis and incorporation of needed stipulations in design and construction projects is recommended:

1. In consultation with the FWS, assess known eagle nests during the reconnaissance/location phase that lasts 1 to 2 years.

Measure ambient conditions, particularly human disturbance in relation to the nesting sequence. Evaluate potential disturbance of nesting eagles by proposed construction techniques, including effects on wind firmness of nesting trees. Consider use of habituation to acclimate the birds to minimize impacts of construction. Include required or recommended procedures in the project environmental document.

2. Incorporate required or recommended procedures into the design phase of the project. Initiate habituation, if necessary, at this time. Identify potential construction disturbances that may significantly alter nesting behavior, thereby halting construction.

3. Clearly list, in the project bid documents, any limitations on construction procedures or timing (as determined in Items 1 and 2). Clearly state conditions under which field monitoring may be required. List any known conditions under which work will or can be modified, curtailed, or rescheduled.

4. During construction, perform field monitoring (using a trained observer) to ensure contractor compliance with stipulations as spelled out in contract bid documents. Where necessary, monitor eagles to track those situations that might require project alteration or shutdown.

5. Summarize field data and notes in a project construction monitoring report. The report should assess the project construction guidelines to minimize disturbance as stipulated in the bid documents and how they were implemented during construction. This report should include recommended changes or improvements for future projects. A copy should be sent to the FWS for their review and comment.

6. Monitor nest use and productivity in succeeding years to confirm the level of construction impacts. Without banding, nest site tenacity is an unknown. However, assuming nest sites are the limiting factor in urban areas, continued use of the site following construction may indicate no appreciable impacts from construction activities.

## CONCLUSIONS

Protecting nesting bald eagles near highway construction projects is not always a simple matter of merely applying the buffer zones and timing constraints as recommended by the FWS. The 100-m buffer zone for general construction, 800-m buffer zone for blasting, and the timing restriction of March through August for active nests are often too restrictive. This is particularly true for eagles nesting in urban areas. In Southeast Alaska, the March through August closure is three-fourths of the average construction season. Unnecessary restrictions on construction timing or techniques can significantly increase project costs.

An array of variables including food supplies, satisfactory nest sites, and innate and learned behavior of individual birds can greatly affect nest site tenacity of any given pair of bald eagles. The greater the nest site tenacity, the less potential disturbance caused by construction activities. In order to address this variability, each nesting pair must be addressed on a case-by-case basis. As demonstrated in the case studies presented, construction can often proceed within the FWS-recommended buffer zones and timing restriction.

A systematic methodology to assess eagles on a case-by-case basis should be developed in consultation with the FWS. This approach to maintaining long-term productivity of eagle nests adjacent to urban construction projects should show good faith intent to abide by the mandates of the Bald Eagle Protection Act. Addressing the potential construction impacts on nesting bald eagles and prescribing mitigation measures in the project National Environmental Policy Act document, plus implementing the agreed-to stipulations to prevent disturbance during construction should also avoid legal action.

Incorporation of realistic, enforceable stipulations in project environmental and construction bid documents in a timely manner is necessary. It would allow the maximum flexibility necessary to schedule highway projects to minimize design and construction costs.

## ACKNOWLEDGMENTS

The author would like to thank Skip Ambrose, Mike Jacobson, and Phil Schempf of the U.S. Fish and Wildlife Service; Bob Ritchie of Alaska Biological Research; Art Dunn, Mike McKinnon, and Van Sundberg of the Alaska Department of Transportation and Public Facilities; and Irv Lloyd of the FHWA for their ideas and helpful criticism. Without Vanda Randolph's patience and proficient typing, preparation of this report would have been a headache. Funding for this project was made available through the FHWA.

## REFERENCES

1. J. I. Hodges. *Evaluation of the 100 Meter Protective Zone for Bald Eagle Nests in Southeast Alaska.* Raptor Management Studies, U.S. Fish and Wildlife Service, Juneau, Alaska, 1982, 11 pp.
2. J. E. Mathisen. Effects of Human Disturbance on Nesting Bald Eagles. *Journal of Wildlife Management*, Vol. 32, No. 1, 1968, pp. 1–6.
3. U.S. Army Corps of Engineers. *The Northern Bald Eagle* (Haliaeetus leucocephalus alaskanus). *A literature survey.* U.S. Army Corps of Engineers, Seattle, Wash., 1979, 86 pp.
4. J. L. Lincer, W. Clark, and M. N. France, Jr. *Working Bibliography of the Bald Eagle.* Raptor Information Center, National Wildlife Federation, Washington, D.C., 1978.
5. B. G. Jueneman and L. D. Frenzel. Habitat Evaluations of Selected Bald Eagle Nest Sites in the Chippewa National Forest. *Trans., 34th American Midwest Fish and Wildlife Conference*, Des Moines, Iowa, 1972, 4 pp.
6. E. E. Bangs, T. N. Bailey, and V. D. Berns. Ecology of Nesting Bald Eagles on the Kenai National Wildlife Refuge, Alaska. *Proc., Symposium and Workshop, Raptor Management and Biology in Alaska and Western Canada*, W. N. Ladd and P. F. Schempf, eds., U.S. Fish and Wildlife Service, Anchorage, Alaska, 1987, pp. 47–54.
7. J. D. Fraser, L. D. Frenzel, and J. E. Mathisen. The Impact of Human Activities on Breeding Bald Eagles in North-Central Minnesota. *Journal of Wildlife Management*, Vol. 49, No. 3, 1985, pp. 585–592.
8. R. G. Anthony and F. B. Isaacs. Characteristics of Bald Eagle Nest Sites in Oregon. *Journal of Wildlife Management*, Vol. 53, No. 1, 1989, pp. 148–159.
9. J. R. Murphy. Nest Site Selection of the Bald Eagle in Yellowstone National Park. *Proc., Utah Academy of Sciences, Arts, and Letters*, Vol. 42, 1965, pp. 261–264.
10. L. I. Retfalvi. *Breeding Behavior and Feeding Habits of the Bald Eagle* (Haliaeetus leucocephalus) on San Juan Island, Washington. M.S. thesis, University of British Columbia, Vancouver, Canada, 1965.
11. F. M. Weekes. A Survey of Bald Eagle Nesting Attempts in Southern Ontario, 1969–73. *Canadian Field Naturalist*, Vol. 88, 1974, pp. 415–419.

12. R. J. Ritchie. *Response of Adult Peregrine Falcons to Experimental and Other Disturbances Along the Trans-Alaska Pipeline System, Sagavanirktok River, Alaska, 1985, 1986.* Alaska Biological Research, Inc., Fairbanks, Alaska, 1987, 92 pp.
13. S. L. Cain. *Nesting Activity Time Budgets of Bald Eagles in Southeast Alaska.* M.S. thesis, University of Montana, Missoula, Mont., 1985, 47 pp.
14. D. K. Garcelon, T. G. Grubb, and S. Porter. Video Surveillance Systems for Monitoring Nesting Raptors. Presented at the 1988 Annual Meeting of the Raptor Research Foundation, Inc., at Minneapolis, Minn., 1988.
15. P. O. Corr. *Bald Eagle* (Haliaeetus leucocephalus alaskanus) *Nesting Related to Forestry in Southeastern Alaska.* M.S. thesis, University of Alaska, College, Alaska, 1974, 144 pp.
16. P. F. Schempf. Raptor Management Studies, U.S. Fish and Wildlife Service, Juneau, Alaska. *Proc., Symposium and Workshop, Raptor Management and Biology in Alaska and Western Canada,* W. N. Ladd and P. F. Schempf, eds., U.S. Fish and Wildlife Service, Anchorage, Alaska, 1982, pp. 12–18.
17. C. Snow. *Southern and Northern Bald Eagle.* Habitat Management Series for Endangered Species, Report 5, U.S. Bureau of Land Management, 58 pp.
18. R. Hayes and D. H. Mossop. The Recovery of an Interior Peregrine Falcon Population in the Northern Yukon Territory. *Proc., Symposium and Workshop, Raptor Management and Biology in Alaska and Western Canada,* W. N. Ladd and P. F. Schempf, eds., U.S. Fish and Wildlife Service, Anchorage, Alaska, 1982, pp. 234–243.
19. M. V. Stalmaster, R. L. Knight, B. L. Holder, and R. J. Anderson. Bald Eagles. In *Management of Wildlife and Fish Habitat in Forests of Western Oregon and Washington,* E. R. Brown, ed., Publication R6-F&WL-192-1985. U.S. Department of Agriculture, Forest Service, Portland, Oreg., 1985, pp. 269–290.
20. W. B. Sidle, L. H. Suring, and J. I. Hodges, Jr. *The Bald Eagle in Southeast Alaska.* Wildlife and Fisheries Habitat Management Notes No. 11, U.S. Forest Service, Alaska Region, Juneau, 29 pp.
21. T. E. Musselman. Concentrations of Bald Eagles on the Mississippi River at Hamilton, Illinois. *Auk,* Vol. 66, 1949, 83 pp.
22. S. K. Sherrod, C. M. White, and F. S. L. Williamson. Biology of the Bald Eagle on Amchitka Island, Alaska. *Living Bird,* Vol. 15, pp. 143–182.
23. M. A. McCollough. *The Post-Fledgling Ecology and Population Dynamics of Bald Eagles in Maine.* Ph.D. dissertation. University of Maine, Orono, 1986, 132 pp.
24. J. G. King, F. C. Robards, and C. J. Lensink. Census of the Bald Eagle Breeding Population in Southeast Alaska. *Journal of Wildlife Management,* Vol. 36, No. 4, 1972, pp. 1292–1295.
25. M. Amaral. Recommended Restrictions for Protection of Peregrine Falcons in Alaska. *Proc., Symposium and Workshop, Raptor Management and Biology in Alaska and Western Canada,* W. N. Ladd and P. F. Schempf, eds., U.S. Fish and Wildlife Service, Anchorage, Alaska, 1982, pp. 217–233.
26. C. M. White and T. L. Thurow. Reproduction of Ferruginous Hawks Exposed to Controlled Disturbance. *The Condor,* Vol. 87, 1985, pp. 12–14.
27. M. J. Jacobson. *The Capture of Alaskan Bald Eagles for Translocation to Other States and Related Productivity Studies—1987.* Raptor Management Studies, U.S. Fish and Wildlife Service, Juneau, Alaska, 1987, 12 pp.
28. J. E. Mathisen, D. E. Sorenson, L. D. Frenzel, and T. C. Dunstan. Management Strategy for Bald Eagles. *Trans., North American Wildlife and National Resources Conference,* Vol. 42, 1977, pp. 86–92.

*Publication of this paper sponsored by Committee on Landscape and Environmental Design.*

# Highway Water Quality Control—Summary of 15 Years of Research

DOUGLAS L. SMITH AND BYRON N. LORD

The year 1989 marked the completion of a major segment of FHWA's environmental research program. More than 15 years of water quality research have been supported by FHWA and state highway agencies. Since the early 1970s, FHWA has supported a four-phase research program in non–point-source pollution from highway runoff. Phase I identified the constituents of highway runoff and developed a data base of highway runoff quality and quantity. Phase 2 identified the sources and migration patterns of highway runoff constituents and further developed the Phase 1 data base. Phase 3 results indicated that highway facilities with low-to-medium average daily traffic (less than 30,000 vehicles per day) exhibited minimal impact on receiving waters. Phase 4 developed a new predictive procedure for estimating the pollutant loadings from highway sources, and identified practical, effective, and implementable mitigation measures to reduce or eliminate the impacts from highway runoff. The need for further environmental research is discussed.

The National Environmental Policy Act of 1969 (Public Law 91–190) requires that all federal agencies prepare environmental impact statements on major federal or federally regulated actions. The Clean Water Act (Public Law 92–500) established a comprehensive national water quality program. The overall objective of this legislation is to report and maintain the chemical, physical, and biological integrity of U.S. waters. The 1977 amendments to the Clean Water Act required federal agencies to cooperate with state and local agencies to develop comprehensive solutions to prevent, reduce, and eliminate pollution.

The growing awareness of the potential threat to the environment by highway runoff and the corresponding lack of information as to the true nature and extent of this threat established the need to initiate a highway water quality research program. FHWA initiated a four-phased cooperative federal and state research and development program to identify and quantify the effects of highway runoff, and to develop measures for protecting the environment from adverse effects. The four phases of the program are as follows:

- Phase 1. Identify and quantify the constituents of highway runoff,
- Phase 2. Identify the sources of the constituents and their migration paths from the highway to the receiving waters,
- Phase 3. Analyze the effects of the constituents in receiving waters, and
- Phase 4. Develop the necessary abatement and treatment methodologies for objectionable constituents.

Federal Highway Administration, Turner-Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, Va. 22101.

## RESULTS OF PHASE 1

The objectives of Phase 1 were to identify and quantify the constituents of highway runoff. In order to accomplish these objectives, a total of 159 storm events were monitored at six sites between the spring of 1976 and September 1977. The data were evaluated for relationships between rainfall and runoff; highway runoff pollutant loadings and variation with time; differences in pollutant characteristics from paved and nonpaved areas; and correlation of pollutants among measured parameters as well as highway operation–related factors. Table 1 presents a summary of the water quality data from Phase 1. The results of Phase 1 are documented in a six-volume report titled *Constituents of Highway Runoff* (*1–6*).

The data generated from the study were used to develop a predictive procedure to provide highway designers and others with a simplified tool to predict the quantity and quality of rainfall-generated highway runoff (*3*). The procedure is made up of four components corresponding to the following functions: rainfall, runoff, pollutant washoff, and constitutent loading. The predictive procedure calculates the volume of runoff given rainfall volume using equations developed from the monitoring data and regression analyses. Because the rainfall-to-runoff relationship is dependent on site characteristics, an equation was developed for each of three basic site types, as follows (*4*):

- Type 1. Totally paved bridges or overpasses (100 percent paved),
- Type 2. Partially paved sections with curbs and inlets along the paved areas (30 to 40 percent paved), and
- Type 3. Rural sites with flush shoulders, grassy ditch conveyances to inlets (20 to 30 percent paved).

## RESULTS OF PHASE 2

The overall objectives of Phase 2 were to identify the sources of highway pollutants and to investigate their deposition and accumulation within the highway system, and their subsequent removal from the highway system to the surrounding environment. The research was also to identify opportunities for pollutant mitigation. The complete findings from Phase 2 are documented in a four-volume report, *Sources and Migration of Highway Runoff Pollutants* (*7–10*).

Although further investigation was required to obtain the sources of some highway pollutants, including pathogenic indicator bacteria, asbestos, and polychlorinated biphenols (PCB), the sources for many others were found to be ade-

TABLE 1 HIGHWAY RUNOFF DATA FROM ALL SIX MONITORING SITES IN PHASE 1 RESEARCH (4)

| | Pollutant concentration mg/l | | Pollutant Loadings lb/ac/event | | Pollutant Loadings lb/ac/in-runoff | |
|---|---|---|---|---|---|---|
| | Ave. | Range | Ave. | Range | Ave. | Range |
| pH | | 6.5 - 8.1 | | | | |
| TS | 1,147 | 145-21,640 | 51.8 | 0.04-535.0 | 260 | 33 - 4,910 |
| SS | 261 | 4 - 1,156 | 14.0 | 0.008-96.0 | 59 | 0.9 - 375 |
| VSS | 77 | 1 - 837 | 3.7 | 0.004-28.2 | 17 | 0.2 - 190 |
| $BOD_5$ | 24 | 2 - 133 | 0.88 | 0.000- 4.1 | 5.4 | 0.5 - 30 |
| TOC | 41 | 5 - 290 | 2.1 | 0,002-11.5 | 9.3 | 1.1 - 66 |
| COD | 14.7 | 5 - 1,058 | 6.9 | 0.004-34.3 | 33 | 1.1 - 240 |
| TKN | 2.99 | 0.1 - 14 | 0.15 | 0.000-1.04 | 0.68 | 0.02 - 3.17 |
| $NO_2$+$NO_3$ | 1.14 | 0.01- 8.4 | .069 | 0.000-0.42 | 0.26 | 0.002- 1.90 |
| $TPO_4$ | 0.79 | 0.05-3.55 | .047 | 0.000-0.36 | 0.18 | 0.011- 0.81 |
| Cl | 386 | 5 -13,300 | 13.0 | 0.008- 329 | 88 | 1.1 - 3,015 |
| Pb | 0.96 | 0.02-13.1 | .058 | 0.000- 4.8 | 0.22 | 0.005- 2.97 |
| Zn | 0.41 | 0.01- 3.4 | .022 | 0.000- 0.8 | 0.093 | 0.002-0.771 |
| Fe | 10.3 | 0.1 -45.0 | 0.50 | 0.000- 3.5 | 2.34 | 0.023- 10.2 |
| Cu | 0.103 | 0.01-0.88 | .0056 | 0.000-.029 | 0.023 | 0.002-0.199 |
| Cd | 0.040 | 0.01-0.40 | .0017 | 0.000-.014 | 0.009 | 0.002-0.091 |
| Cr | 0.040 | 0.01-0.14 | .0028 | 0.000-.029 | 0.009 | 0.002-0.032 |
| Hg $x10^{-3}$ | 3.22 | 0.13-67.0 | 00059 | 0.000-.002 | 0.730 | 0.029- 15.2 |
| Ni | 9.92 | 0.1 - 49 | 0.27 | 0.007-1.33 | 2.25 | 0.023- 11.2 |
| TVS | 242 | 26 -1,522 | 9.34 | 0.01 -44.0 | 55 | 5.98 - 345 |

quately documented in the literature. Because of the detailed nature of the pollutant source studies, the majority of the monitoring was conducted at the Milwaukee I-94 site. Table 2 presents some of the common constituents of highway runoff and their identified sources.

Data were also collected at all monitoring sites to evaluate the qualitative and quantitative aspects of background pollutant loading to the highway system, pollutants originating from the highway system, and the mechanism of pollutant dispersion within and transfer out of the highway system. Variables that affect pollutant deposition, accumulation, and removal were measured to facilitate data evaluation. These variables include traffic characteristics, maintenance activities, and climatic conditions. Field studies were conducted for a minimum 12-month period on four sites to evaluate seasonal effects on a number of parameters. The elements of the field data collection are as follows (7):

- Atmospheric deposition,
- Total suspended particulate,
- Saltation (bouncing particulate),
- Highway surface loads (monitored through sweeping and flushing studies),
- Runoff quantity and quality,
- Ground water percolation (monitored by lysimeters),
- Soil and vegetation,
- Traffic characteristics,
- Highway maintenance activities, and
- Climatological conditions.

Characteristics of the four monitoring sites are presented in Table 3.

Background bulk precipitation data (wet and dry deposition) indicate total particulate matter (TPM) loadings, solids in the atmosphere that are washed out by rainfall or drop out, were approximately four times higher at the urban sites than at the rural sites. Similarly, background metals deposition was higher at the urban sites. The data also indicated dry deposition (dust fall) is a more important source of metals than wet deposition (rain).

Runoff data indicated the highway system has a large capacity to buffer the runoff of acid precipitation. The prevalence of acid rain in the United States and the apparent ability of highway systems to buffer this acid rain may have important implications with regard to pollutant migration from the highway through areas adjacent to the highway. The solubility of metals is a function of pH, the quantity of anionic complexing agents, and the organic matter present. At reduced pH values, soluble metals would be easier to remove from the highway surface, would tend to migrate further, and would be more accessible for bioaccumulation. The significant buffering capacity of the highway system serves to minimize this potential.

The research showed that the atmospheric deposition of highway-generated TPM and associated metals onto areas adjacent to the highway is related to average daily traffic (ADT), wind speed and direction, available surface load, and terrain and landscape features. The study concluded that the impact area for highway-generated TPM was approximately 35 m from the edge of pavement for urban sites and 15 m for rural sites.

Runoff from the paved and unpaved areas were segregated to determine pollutant loadings leaving the highway drainage system and to develop insights into the hydraulics of pollutant movement. At sites with curbs and gutters, the contribution of the unpaved area to the total pollutant constituent load removed by runoff was negligible. At sites with flush shoulders, the unpaved area contributed approximately 17 percent of the total load for most constituents. This outcome was the apparent result of solids and pollutants associated with the solids accumulated in the distress and median lanes becoming trapped against the curbs and available to be picked up by runoff from the pavement. In the noncurbed areas, the solids are not trapped and are free to be dispersed into the nonpaved areas. Although the flush shouldered areas showed a higher

TABLE 2 HIGHWAY RUNOFF CONSTITUENTS AND THEIR PRIMARY SOURCES (7)

| Constituents | Primary Sources |
|---|---|
| Particulate | Pavement wear, vehicles, atmosphere, maintenance |
| Nitrogen, Phosphorus | Atmosphere, roadside fertilizer application |
| Lead | Leaded gasoline (auto exhaust), tire wear (lead oxide filler material), lubricating oil and grease, bearing wear |
| Zinc | Tire wear (filler material), motor oil (stabilizing additive), grease |
| Iron | Auto body rust, steel highway structures, (guard rails, etc.), moving engine parts |
| Copper | Metal plating, bearing and bushing wear, moving engine parts, brake lining wear, fungicides and insecticides |
| Cadmium | Tire wear (filler material), insecticide application |
| Chromium | Metal plating, moving engine parts, break lining wear |
| Nickel | Diesel fuel and gasoline (exhaust), lubricating oil, metal plating, bushing wear, brake lining wear, asphalt paving |
| Manganese | Moving engine parts |
| Cyanide | Anticakeing compounds (ferric ferrocyanide, sodium ferrocyanide, yellow prussiate of soda) used to keep deicing salt granular |
| Sodium/Calcium, Chloride | Deicing salts |
| Sulphate | Roadway beds, fuel, deicing salts |
| Petroleum | Spills, leaks or blow-by of motor lubricants, antifreeze and hydraulic fluids, asphalt surface leachate |
| PCB | Spraying of highway rights-of-way, background atmospheric deposition, PCB catalyst in synthetic tires |

proportion of the pollutant load coming from the nonpaved area, the total pollutant loads were significantly smaller.

Soils data indicated metals and sodium concentrations were located in the top soil layers (upper 10 cm). Concentrations were also highest adjacent to the highway and decreased with distance from the highway. Chlorides did not show this gradient. Lysimeter data indicated that chlorides were leached from the upper soil layer shortly after the spring thaw. Uptake of metals and sodium by vegetation was generally related to concentration of these constituents in the topsoil layer. The vegetation and soils data indicated that normal ecosystem processes might be affected within the first couple of meters adjacent to the highway, but beyond this distance no impacts were detected.

## RESULTS OF PHASE 3

The next step, after identifying the constituents of highway runoff, their sources, and their migration paths, was to determine the magnitude and extent of the impacts of highway stormwater runoff on receiving waters and to provide guidance for assessing impacts caused by highway stormwater runoff. An extensive field monitoring and laboratory analysis program was conducted at three sites: two streams and a lake. Results indicated that for highway facilities with low-to-medium ADT (less than 30,000 vehicles per day), there is minimal impact to receiving waters and their associated floral and faunal communities. Results of this research are documented in a five-volume report, *Effects of Highway Runoff on Receiving Waters* (*11–15*).

Acute-toxicity bioassays were performed using undiluted runoff from highways with ADT ranging from 12,000 to 120,000 vehicles per day. These bioassays were conducted to simulate worse-case shock loadings on the receiving waters for durations of no more than several days. Certain assumptions were implicit in this assessment: (a) the quality of the receiving water may be temporarily degraded, but will rapidly return to its previous state; (b) detrimental substances in the runoff water are flushed out of the receiving waters and do not linger; and (c) detrimental effects on aquatic biota are caused by direct toxicity, not indirect or delayed effects. Bioassays were run on an alga (*Selenastrum capricornutum*), water flea (*Daphnia magna*), amphipod (*Gammarus pseudolimnaeus*), isopod (*Asellus intermedius*), mayfly (*Hexagenia* sp.), and flathead minnow (*Pimephales promelas*).

Undiluted highway runoff was not acutely toxic to the flathead minnow. Fish exposed to the runoff water did, however,

TABLE 3 CHARACTERISTICS OF SITES IN PHASE 2 RESEARCH (7)

| Location | Type | ADT Vehicles/ Day | Precipitation inches/year Total | Snow-Fall | Acres Total | Paved | % Paved | Surface Type | Hwy Length ft | Number of Travel Lanes | Type of Section | Curb/ Barrier |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Milwaukee Wisconsin I-94 | Urban | 116,000 | 30 | 45 | 7.60 | 4.90 | 64 | Asphalt | 1,373 | 8 | cut/fill | yes |
| Sacramento California US-50 | Urban | 85,900 | 17 | 0 | 2.45 | 2.01 | 82 | Concrete | 1,400 | 8 | at grade | yes |
| Harrisburg Pennsylvania I-81 | Rural | 27,800 | 38 | 35 | 2.81 | 1.05 | 45 | Concrete | 1,345 | 4 | cut/ at grade | no |
| Efland N.Carolina I-85 | Rural | 25,000 | 41 | 9 | 2.49 | 1.27 | 51 | Asphalt | 1,025 | 4 | at grade | no |

demonstrate some sluggishness, implying some stress. The isopod was insensitive to exposure to undiluted highway runoff. The amphipod demonstrated some sensitivity to exposure to undiluted runoff from the 12,000-ADT site. Of the observed mortality, no more than 40 percent could be attributed to the direct toxicity of the water. The mayfly nymphs showed slight sensitivity to undiluted highway runoff. Exposure to the 120,000-ADT undiluted runoff did not affect the nymphs from hatching into adults. The *Daphnia* was not sensitive to the 12,000-ADT undiluted runoff in a 96-hr flowthrough assay, nor to 120,000-ADT undiluted runoff in a 48-hr static test. Algal assays, a long-term chronic toxicity test, demonstrated adverse effects of undiluted runoff water on algal growth. The results of the assays using 120,000-ADT water demonstrated a probable heavy metal stress on algae.

As was mentioned, the bioassays represented a worst-case scenario. Under normal conditions, highway runoff is significantly diluted on entering the receiving waters. Therefore, these results indicate that highway runoff would have minimal (if any) toxic effect under normal conditions.

The results from the field monitoring sites indicated that the highway right-of-way contributed a small percentage (0.03 to 5 percent, depending on the site location and constituent being measured) of the total watershed load. Water quality impacts from highway runoff during storm events were not apparent. At the Lower Neahbin Lake site, direct discharge of highway runoff from the bridge deck caused localized increases in metals and salts in near shore sediments and cattails (*Typha* sp.). Metal concentrations found in the cattails were found to decrease to background levels within 65 ft of the input point. The ultimate impact of these accumulations is uncertain.

## RESULTS OF PHASE 4

The completion of Phase 4 during 1989 brought to a close this focused research effort on highway runoff water quality. This phase was accomplished through three research efforts. The first study, documented in a four-volume report, *Management Practices for Mitigation of Highway Stormwater Runoff Pollution* (*16–19*), identified practical, effective, and implementable mitigation measures to reduce or eliminate the impacts of highway runoff. These measures were to serve as interim guidance until the completion of the research program.

A second study used the vast amount of data collected through FHWA contract research, federally assisted state studies, U.S. Environmental Protection Agency's Nationwide Urban Runoff Program, and state highway agency sponsored research to develop an improved prediction model to estimate pollutant loadings from the highway. This research, documented in a four-volume report, *Evaluation of Pollutant Impacts from Highway Stormwater Runoff* (*20–23*), also includes a computerized version of the model. The results are based on 993 individual storm events at 31 highway sites in 11 states. Impact prediction is based on a methodology previously developed and applied to urban runoff and adapted for highway runoff application (*24*).

The final study in Phase 4 evaluated the use of retention, detention, and overland flow systems as potential mitigation measures. As an initial step in the study, interim design guidelines were developed (*25*). These design guidelines were then refined on the basis of the results of laboratory and field evaluations and are documented in a two-volume report, *Retention, Detention, and Overland Flow for Pollutant Removal from Highway Stormwater Runoff* (*26,27*).

The five management measures that were considered cost effective for pollutant removal from highway runoff were as follows (*27*):

- Vegetation controls,
- Wet detention basins,
- Dry extended detention basins,
- Infiltration systems (also called "retention measures"), and
- Wetlands.

In addition to discussing and providing specific design guidance for the construction of these five measures, the report (*27*) also discusses the following effective nonstructural measures to reduce pollutant loadings from highway runoff:

- Curb elimination,
- Litter control,
- Deicing chemical use management,
- Pesticide and herbicide use management,
- Reduced direct discharge,
- Reduced runoff velocity, and
- Establishment and maintenance of vegetation.

Common practices that are ineffective at reducing pollutant loads include

- Street cleaning,
- Catch basins,
- Porous pavements, and
- Filtration devices for sediment control.

These practices were developed on the basis of actual field monitoring or obtained from published literature.

## RELATED RESEARCH

Although the four-phased water quality research program just described was the central focus of the FHWA water quality program, other related research and implementation activities have been conducted and are currently underway that are directly and indirectly related. One study, which was a direct spinoff of the program, investigated the impacts of highway maintenance activities on water quality. The resulting four-volume report (*28–31*) identifies commonly used maintenance practices and describes each in terms of its potential for causing adverse impacts to water quality. The study identified the following six practices that could have an impact if proper measures are not taken:

- Repairing slopes, slips, and slides;
- Cleaning ditches, channels, and drainage structures;
- Bridge painting;
- Substructure repair; and
- Chemical vegetation control.

The study also provides guidelines on available methods of avoiding or mitigating these potential impacts.

Early in the program, FHWA sponsored the development of a water quality training course, offered through the National Highway Institute (NHI). In 1985, the water quality course was updated. One of the most significant products of the new course is the student workbook (*32*). It provides, in one volume, a compendium of the FHWA water quality research. The only information lacking from this text are the final results from the retention, detention, and overland flow research, and the new pollutant discharge model. The text does, however, excerpt material from the earlier interim design guidelines (*25*).

Another area of significant research has been the development of calcium magnesium acetate (CMA) as an alternative deicer. This chemical, while more costly than salt, has been shown to be effective as a deicer and to have less of an impact on the environment. The selective use of CMA in environmentally critical areas, coupled with improved storm detection and prediction, pavement condition, and winter maintenance management, can greatly aid in reducing highway runoff pollution in the snow belt regions of the country.

The last technically related area that should be discussed is the FHWA wetlands research program. To date, this program and the water quality research program have been conducted in parallel, investigating two separate but related topics of concern. One of the most significant accomplishments by FHWA in wetlands research is the development of the report, *A Method for Wetland Functional Assessment* (*33,34*), and the subsequent cooperative agreement with the U.S. Army Engineers in developing an enhanced assessment method, *The Wetland Evaluation Technique*, Version 2.0 (*35*).

## FUTURE OF THE FHWA WATER QUALITY RESEARCH AND DEVELOPMENT PROGRAM

Although the four-phased research effort has resulted in considerable information and a large number of projects that now give the highway agencies the tools they need to meet the immediate challenge, more work needs to be done. Since the original work on the constituents of highway runoff, there have been a number of changes made in the fuels used in automobiles and in automobile technology. There has not been any recent research to determine how these changes may have affected highway runoff water quality and the effects to receiving waters.

With the current concern for protecting our wetland resource and the likelihood that the concept of no net loss of wetlands will become national policy, to gain a fuller understanding of how highway runoff may affect wetland systems is vital. Although some research has been conducted (*36*), a focused research effort is needed to determine whether it is feasible and prudent to use natural wetlands or wetlands constructed for wetland mitigation as a means of treating highway runoff. Under what circumstances this practice might be detrimental to the other functions of the wetland is not known.

Another area of major concern is the control of hazardous materials spilled within the highway rights-of-way. FHWA is sponsoring research to aid in determining the need for protective systems and available technology to control spills from impacting ground and surface waters. However, little if any work has been done on developing cost-effective systems that will do the job.

As part of FHWA plans for an expanded research and development budget beginning in FY 1992, provision is being made for a significantly expanded environmental research program. If the program becomes a reality, FHWA will continue to assist the state highway agencies in providing the tools needed to protect our nation's waters.

## REFERENCES

1. M. K. Gupta, R. W. Agnew, and N. P. Kobriger. *Constituents of Highway Runoff: Volume I, State of the Art Report.* Report FHWA/RD-81/042. FHWA, U.S. Department of Transportation, 1981.
2. M. K. Gupta, R. W. Agnew, and T. L. Meinholz. *Constituents of Highway Runoff: Volume II, Procedural Manual for Monitoring Highway Runoff.* Report FHWA/RD-81/043. FHWA, U.S. Department of Transportation, 1981.

3. N. P. Kobriger, T. L. Meinholz, M. K. Gupta, and R. W. Agnew. *Constituents of Highway Runoff: Volume III, Predictive Procedure for Determining Pollutant Characteristics in Highway Runoff.* Report FHWA/RD-81/044. FHWA, U.S. Department of Transportation, 1981.
4. M. K. Gupta, R. W. Agnew, D. Gruber, and W. Kreutzberger. *Constituents of Highway Runoff: Volume IV, Characteristics of Runoff from Operating Highways, Research Report.* Report FHWA/RD-81/045. FHWA, U.S. Department of Transportation, 1981.
5. N. P. Kobriger and T. M. Krischan. *Constituents of Highway Runoff: Volume V, Highway Runoff Data Storage Program and Computer User's Manual.* Report FHWA/RD-81/046. FHWA, U.S. Department of Transportation, 1981.
6. M. K. Gupta, R. W. Agnew, and N. P. Kobriger. *Constituents of Highway Runoff: Volume VI, Executive Summary.* Report FHWA/RD-81/047. FHWA, U.S. Department of Transportation, 1981.
7. N. P. Kobriger. *Sources and Migration of Highway Pollutants: Volume I, Executive Summary.* Report FHWA/RD-84/057. FHWA, U.S. Department of Transportation, 1984.
8. N. P. Kobriger and M. K. Gupta. *Sources and Migration of Highway Pollutants: Volume II, Methods.* Report FHWA/RD-84/058. FHWA, U.S. Department of Transportation, 1984.
9. N. P. Kobriger and A. Geinopolos. *Sources and Migration of Highway Pollutants: Volume III, Research Report.* Report FHWA/RD-84/059. FHWA, U.S. Department of Transportation, 1984.
10. N. P. Kobriger. *Sources and Migration of Highway Pollutants: Volume IV, Appendix.* Report FHWA/RD-84/060. FHWA, U.S. Department of Transportation, 1984.
11. T. V. Dupius et al. *Effects of Highway Runoff on Receiving Waters: Volume I, Executive Summary.* Report FHWA/RD-84/062. FHWA, U.S. Department of Transportation, 1985.
12. T. V. Dupius, J. Kastewr, P. Bertram, J. Meyer, M. Smith, and N. P. Kobriger. *Effects of Highway Runoff on Receiving Waters: Volume II, Research Report.* Report FHWA/RD-84/063. FHWA, U.S. Department of Transportation, 1985.
13. T. V. Dupius, N. P. Kobriger, W. K. Kreutzberger, and V. Trippi. *Effects of Highway Runoff on Receiving Waters: Volume III, Resource Document for Environmental Assessments.* Report FHWA/RD-84/064. FHWA, U.S. Department of Transportation, 1985.
14. T. V. Dupius and N. P. Kobriger. *Effects of Highway Runoff on Receiving Waters: Volume IV, Procedural Guidelines for Environmental Assessments.* Report FHWA/RD-84/065. FHWA, U.S. Department of Transportation, 1985.
15. T. V. Dupius, T. Kreutzberger, J. Kaster, and T. Harris. *Effects of Highway Runoff on Receiving Waters: Volume V, Guidelines for Conducting Field Studies.* Report FHWA/RD-84/066. FHWA, U.S. Department of Transportation, 1985.
16. C. W. Burch, F. Johnson, and B. Maestri. *Management Practices for Mitigation of Highway Stormwater Runoff Pollution: Volume I, Guidelines.* Report FHWA/RD-85/001. FHWA, U.S. Department of Transportation, 1985.
17. C. W. Burch, F. Johnson, and B. Maestri. *Management Practices for Mitigation of Highway Stormwater Runoff Pollution: Volume II, Literature Review.* Report FHWA/RD-85/002. FHWA, U.S. Department of Transportation, 1985.
18. B. Maestri, F. Johnson, and C. W. Burch. *Management Practices for Mitigation of Highway Stormwater Runoff Pollution: Volume III, Research Report.* Report FHWA/RD-85/003. FHWA, U.S. Department of Transportation, 1985.
19. C. W. Burch, F. Johnson, and B. Maestri. *Management Practices for Mitigation of Highway Stormwater Runoff Pollution: Volume IV, Executive Summary.* Report FHWA/RD-85/004. FHWA, U.S. Department of Transportation, 1985.
20. E. D. Driscoll, P. E. Shelley, and E. W. Strecker. *Evaluation of Pollutant Loadings from Highway Stormwater Runoff: Design Procedures.* Report FHWA/RD-88/006. FHWA, U.S. Department of Transportation, 1988.
21. E. D. Driscoll, P. E. Shelley, and E. W. Strecker. *Evaluation of Pollutant Loadings from Highway Stormwater Runoff: User's Guide for Interactive Computer Implementation of Design Procedure.* Report FHWA/RD-88/007. FHWA, U.S. Department of Transportation, 1988.
22. E. D. Driscoll, P. E. Shelley, and E. W. Strecker. *Evaluation of Pollutant Loadings from Highway Stormwater Runoff: Analytical Investigations of Pollutant Loadings and Impacts from Highway Stormwater Runoff.* Report FHWA/RD-88/008. FHWA, U.S. Department of Transportation, 1988.
23. E. D. Driscoll, P. E. Shelley, and E. W. Strecker. *Evaluation of Pollutant Loadings from Highway Stormwater Runoff: Data Appendix.* Report FHWA/RD-88/009. FHWA, U.S. Department of Transportation, 1988.
24. Environmental Protection Agency. *Final Report of the Nationwide Urban Runoff Program.* U.S. Environmental Protection Agency, 1983.
25. M. E. Dorman, J. Hartigan, F. Johnson, and B. Maestri. *Retention, Detention, and Overland Flow for Pollution Removal from Highway Stormwater Runoff: Interim Guidelines for Management Measures.* Report FHWA/RD-87/056. FHWA, U.S. Department of Transportation, 1988.
26. M. E. Dorman, J. Hartigan, R. F. Steg, and T. Quasebarth. *Retention, Detention, and Overland Flow for Pollution Removal from Highway Stormwater Runoff: Volume I, Research Report.* Report FHWA/RD-89/202. FHWA, U.S. Department of Transportation, 1989.
27. J. P. Hartigan, T. S. George, T. F. Quasebarth, and M. E. Dorman. *Retention, Detention, and Overland Flow for Pollution Removal from Highway Stormwater Runoff: Volume II, Design Guidelines.* Report FHWA/RD-89/203. FHWA, U.S. Department of Transportation, 1989.
28. A. D. Kramme, R. G. Rolan, and L. A. Smith. *Highway Maintenance Impacts to Water Quality—Executive Summary.* Report FHWA/RD-85/057. FHWA, U.S. Department of Transportation, 1985.
29. A. D. Kramme, R. G. Rolan, and L. A. Smith. *Investigations of Impacts of Selected Highway Maintenance Practices on Water Quality.* Report FHWA/RD-85/058. FHWA, U.S. Department of Transportation, 1985.
30. A. D. Kramme, R. G. Rolan, and L. A. Smith. *A Reference Manual for Assessing Water Quality Impacts from Highway Maintenance Practices.* Report FHWA/RD-85/059. FHWA, U.S. Department of Transportation, 1985.
31. A. D. Kramme, R. G. Rolan, and L. A. Smith. *Guidelines Manual for Minimizing Water Quality Impacts from Highway Maintenance Practices.* Report FHWA/RD-85/060. FHWA, U.S. Department of Transportation, 1985.
32. T. Denbow. *Highway Runoff Water Quality Training Course—Student Workbook.* National Highway Institute, FHWA, U.S. Department of Transportation, 1986.
33. P. R. Adamus and L. T. Stockwell. *A Method for Wetland Functional Assessment: Volume I, Critical Review and Evaluation Concepts.* Report FHWA-IP-82-23. FHWA, U.S. Department of Transportation, 1983.
34. P. R. Adamus. *A Method for Wetland Functional Assessment: Volume II, FHWA Assessment Method.* Report FHWA-IP-82-24. FHWA, U.S. Department of Transportation, 1983.
35. P. R. Adamus, E. J. Clairain, R. D. Smith, and R. E. Young. *Wetland Evaluation Technique (WET): Volume II, Methodology.* Report FHWA-IP-80-29. U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss., 1987.
36. N. P. Kobriger, T. V. Dupius, W. A. Kreutzberger, F. Stearns, G. Guntenspergen, and J. R. Keough. *Effects of Highway Runoff on Wetlands.* Final Report, NCHRP Project 25-1, TRB, National Research Council, Washington, D.C., 1984.

*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# Development of Planned Management Roadside Design for Urban Highways in Illinois

CHARLES H. GOUVEIA

Roadsides on the urban expressway system were originally designed as parkways requiring intensive management for their upkeep. When the ability to perform those management practices was drastically reduced because of economics, fuel shortages, and other factors, the approach to roadside design had to be totally reevaluated. As a result of that reevaluation, planned-management roadside design, a maintenance-based design approach, was developed. This concept used landscape and turf design based on preplanned management of each portion of the right-of-way. Salt-resistant turf, native grasses, forbs, native trees, and shrubs were extensively used in the implementation of this concept. The goals of planned-management roadside design include reduced construction costs, reduced maintenance efforts, and improved public acceptance.

Planned-management roadside design in Illinois was developed in response to a major shift in roadside maintenance priorities away from the high order of maintenance originally conceived for the system.

When the Interstate system was developed in the 1950s, the available role models for roadside design and maintenance on this type of facility were the parkway systems of the eastern United States and Germany. Many of these early expressways were literally highways in a park designed to provide a pleasant driving experience visually as well as a safe and rapid trip, particularly for tourist traffic. The developing midwestern Interstate system, although more utilitarian in concept, borrowed the parkway style in its roadside design and maintenance. This style dictated park-like plantings and mowed weed-free turf, a generally manicured landscape worthy of any front lawn or city park.

Maintaining the appearance of such park-like roadsides required considerable input of resources. Frequent mowings were required, including the associated hand trimming and hand mowing. Herbicide application and tree spraying were performed extensively, as were hand pruning, fertilization, and cultivation of planting beds. Landscape designs of this period were planned to be managed in this intense manner, and staffing and other resources were adequately committed for the purpose.

Beginning in the late 1960s, rapidly escalating inflation had begun to increase the cost of manpower. As a result, budgetary limitations reduced the number of personnel available to do maintenance work. The fuel shortage of the early 1970s mandated an effort to conserve fuel for essential activities. The overall need to economize through tightened budgets, restricted fuel, and reduced manpower forced substantial cuts in nonessential activities. Roadside maintenance was deeply affected by these reductions, resulting in an inadequate maintenance policy on the roadside.

When care was withdrawn from the intensively managed landscape, a number of adverse impacts were manifested. When fertilization, regular mowings, and chemical weed control were terminated, the bluegrass turf began to lose vigor, allowing aggressive weeds, annual grasses, and brush to invade. The increased use of deicing salts adversely affected turf and trees, and without repairs and proper care erosion increased and plants declined noticeably. The lack of maintenance resulted in a deterioration of appearance, which signaled to the public that dumping and littering on the right of way was an acceptable activity. This perception, coupled with a reduction in the litter control effort, caused an increase in trash and litter. Taken together, all of these impacts contributed to an extremely poor appearance along the roadside.

In the late 1970s, plans were being advanced for a major rehabilitation project on Interstate 94, the Edens Expressway, one of the first parkway-style freeway routes in the Chicago metropolitan area, originally constructed in the 1950s.

This project involved complete pavement removal and replacement. All staging areas, including sites for processing the old concrete pavement into reusable granular material, would have to be located on the right-of-way because of the developed nature of the adjacent properties. The combined result of reconstruction and staging activities would thoroughly disrupt the majority of the existing roadside landscape, which would require complete restoration.

Previous parkway-style roadside design took into account concerns such as aesthetics, screening from view, texture, and color, but assumed that all necessary roadside maintenance would be provided for the entire right-of-way. Currently, the effort expended on postconstruction roadside maintenance is expected to be minimal. It is therefore necessary to consider future site management as the controlling element in any roadside design.

A concept was developed for maintenance-based roadside design that allowed the proposed future management of an area of roadside to guide the selection of the type of turf and plantings to be established there. To be successful, the concept, planned-management roadside design, had to produce a better-quality end result at an equal or lower construction cost than previous treatments. A degree of public acceptance of the roadside appearance and a lower input of maintenance

Illinois Department of Transportation, District One, 201 West Center Court, Schaumburg, Ill. 60196.

resources than required in previous designs were other necessary requirements.

In order to implement the concept, the Landscape Architects of the Illinois Department of Transportation (IDOT) conducted a site analysis to determine which roadside areas needed regular management, occasional management, or highly infrequent management to preserve the aesthetic and operational characteristics of the highway facility. When the analysis was completed, various plant materials were selected that would be characteristic of the desirable properties needed in each management area to produce an appearance that is acceptable to the public and maintenance at a low cost.

The turf immediately adjacent to the roadway is normally the most intensively managed part of the right-of-way. This portion is normally mowed from one to four times per year for safety, sight distance, litter control, and drainage considerations. The area between the edge of the pavement and the ditch line is also the area of greatest salt concentration. This observation has been confirmed by studies done by the IDOT Environmental Resources Unit that show typical salt-laden spray falling within 20 to 25 ft of the pavement edge. This portion of the right-of-way required a turf mixture that would withstand variable salt concentrations, infrequent mowing, traffic damage, and general lack of care, yet would still maintain an acceptable appearance and resist erosion.

The varieties of grass used in this mix were identified as a result of salt tolerance research done by Howard Kaerwer of the Northup King Company (unpublished), to find companion grasses for salt grass (Fults Puccinella Distans or Fults alkali grass) in a salt-tolerant turf mix. Turf grasses found to have high salt resistance in that study were Dawson creeping red fescue; Kentucky 31 tall fescue; Scaldis hard fescue; Galway turf-type tall fescue; buffalo grass; blue gramma grass; Delray perennial rye grass; and Fults alkali grass. These grasses were formulated into the basic mixture, used beginning in 1979 on the Edens Expressway, which is now known as "Class 2A seeding" in Illinois' *Standard Specifications for Road and Bridge Construction*, 1988 edition. This mixture consists of the following components:

| *Variety* | *Amount (lb/acre)* |
|---|---|
| Tall fescue | 30 |
| Delray perennial rye grass | 10 |
| Dawson creeping red fescue | 10 |
| Scaldis hard fescue | 10 |
| Fults alkali grass | 30 |

Another variation of the basic salt-tolerant mix is also currently in use: a low-maintenance mix, known as Class 2B seeding, that has higher seeding rates of buffalo grass and Dawson red fescue, and no tall fescue, as follows:

| *Variety* | *Amount (lb/acre)* |
|---|---|
| Buffalo grass | 40 |
| Fults alkali grass | 20 |
| Scaldis hard fescue | 10 |
| Dawson creeping red fescue | 10 |
| Delray perennial rye grass | 10 |
| Blue gramma grass | 10 |

This mix produces a short grass under minimal maintenance. Also available is a salt-tolerant sod for boulevards and parking lots where immediate finished appearance and salt tolerance are desired. This sod, containing turf-type tall fescue, is now available on the commercial market, blends well with bluegrass sod, and retains good color during drought stress periods. It consists of the following:

| *Variety* | *Percent of Mix by Weight (%)* |
|---|---|
| Buffalo grass | 30 |
| Galway fineleaf tall fescue | 20 |
| Dawson creeping red fescue | 15 |
| Scaldis hard fescue | 15 |
| Rugby Kentucky bluegrass | 5 |
| Fults alkali grass | 15 |

TABLE 1 COMPARISON OF TURF COSTS BY BID PRICES ($/ACRE), 1989

| Item | Sodding, Salt-Tolerant | Sodding |
|---|---|---|
| Sodding | 16,068.80 | 14,520.00 |
| Topsoil | 5,711.20 | 5,711.20 |
| Fertilizer | 192.60 | 192.60 |
| | Seeding, Salt-Tolerant | Seeding, Native Grasses and Forbs |
| Seeding | 1,208.00 | 4,984.00 |
| Topsoil | 5,711.20 | -0- |
| Fertilizer | 192.60 | -0- |
| Mulch[a] | 4,501.20 | 825.00 |
| Mow stakes | -0- | 42.50 |
| Total | 11,613.00 | 5,851.50 |

[a]Excelsior blanket normally used.

In those right-of-way areas beyond the normal mowing limits, plantings are desired that will reduce routine maintenance to near zero, but which will be acceptable in appearance both to the traveling public and to the immediate neighbors. This goal is accomplished by using various mixtures of native forb and wildflower seed, along with native grasses and wildflower plants, for a combination of utility and natural beauty.

The IDOT Class 4 (modified) native grass mixture consists of the following:

| *Variety* | *Pure Live Seed (lb/acre)* |
|---|---|
| Big blue stem | 5 |
| Indian grass | 5 |
| Little blue stem | 15 |
| Side oats gramma | 5 |
| Perennial rye grass | 20 |

The IDOT Class 5 (modified) native forb and annual wildflower mixture consists of the following:

Annuals. One pound per acre total, not to exceed 25 percent of any species.

- *Coreopsis lanceolata*—lance-leaved coreopsis
- *Gaillardia pulchella*—Indian blanket
- *Ratibida columnaris*—upright prairie coneflower
- *Rudbeckia hirta* triploid—black-eyed Susan
- *Hesperis matronalis*—dames rocket
- *Chrysanthemum leucanthemum Pinnatifidum*—oxeye daisy

Forbs. The total mixture of forbs shall be applied at the rate of 10 lb/acre and the mix shall consist of not fewer than 25 of the following species:

- *Amorpha canescens*—lead plant and innoculant*
- *Anemone cylindrica*—thimbleweed
- *Asclepias tuberosa*—butterfly milkweed
- *Aster azureus*—sky blue aster
- *Aster laevis*—smooth aster
- *Aster novae angliae*—New England aster
- *Baptisia leucopaea*—cream Baptisia and innoculant*
- *Ceanothus americanus*—New Jersey tea
- *Cirsium hillii*—hills prairie thistle
- *Coreopsis plamata*—prairie coreopsis**
- *Echinacaea pallida*—pale purple coneflower
- *Erynigium yuccifolium*—rattlesnake master
- *Heuchera richardsonii*—alum root
- *Liatris aspera*—rough blazing star
- *Liatris pycnostachy*—prairie blazing star
- *Monarda fistulosa*—prairie bergamont
- *Parthenium integrifolium*—prairie quinine
- *Pedicularis canadensis*—prairie betony
- *Penstemon grandiflora*—large penstanmor
- *Petalostemum candidum*—white prairie clover and innoculant*
- *Petalostemum purpureum*—purple prairie clover and innoculant*
- *Potentilla arguta*—prairie cinquefoil
- *Ratibida pinnata*—prairie yellow coneflower**
- *Rubeckia subtomentosa*—sweet coneflower
- *Rudbeckia hirta*—black-eyed Susan
- *Siliphium laciniatum*—compass plant
- *Silphlum terebinthinaceum*—prairie dock
- *Solidago rigida*—stiff goldenrod
- *Tradescantia ohiensis*—spider wart
- *Veronicastrum virginicum*—culvers root

As indicated by a single asterisk, all legume seeds shall be innoculated with the proper *rhizabium* bacteria innoculum in the amount and manner required by the manufacturer of the innoculant before sowing or mixing with other seeds for sowing. The mix shall consist of not less than 10 percent of each species indicated with a double asterisk. The prairie forbs seed mixture shall not contain more than 10 percent of any one kind of seed.

Most native plants and wildflowers require no topsoil or fertilizer to establish, and are acclimated to weather conditions in the midwest. Because native forbs and grasses take 3 to 5 years to establish, annual wildflowers are included in the mixtures to increase public acceptance of the long-grass areas in the first few seasons. Information signing and selective mowing stakes are used to delineate the unmowed areas and to ensure public acceptance. The mowing lines are established on the plans to use the unmowed and wildflower areas as design elements. These areas will appear as well-defined, intentional shapes and forms in the landscape.

In areas not visible to the public, native grasses are often used alone and can provide a weed-free, nearly maintenance-free ground cover that will thrive on a wide variety of soil conditions and provide excellent erosion control. In highly visible areas, additional wildflowers are seeded, or placed as plants and bulbs, to lend color to the roadside and further enhance the appearance of the roadway. All of these techniques are intended to reduce or eliminate the need for mowing, erosion control, chemical application, and other maintenance from large areas of the roadside, and to reduce construction costs by using a method that is visually and environmentally satisfactory (see Table 1).

Many of the roadside management plans include ornamental plantings of trees, shrubs, and seedlings in appropriate areas. They are used as screening of and from adjacent properties, as reforestation for reduction of maintenance, and as landscape features that lend interest to the roadside. In 1981, student aids compared the original planting plans for various expressway projects from the late 1950s and early 1960s with current conditions. Noted were factors of survivability, growth, and general health. A short list of plants that could survive roadside conditions and lack of maintenance was then compiled, as follows:

- Thornless Honeylocust
- Hackberry
- Norway maple
- Green ash
- Crabapple
- Cockspur hawthorn
- Amur maple
- Zabel honeysuckle
- Common lilac

This list has since been amended because of availability of more native plants and improved establishment technology, as follows:

*Shade Trees*

- Honeylocust (several varieties)
- Ash (green and white), Hesse ash
- Hackberry
- Ginkgo
- Robusta poplar
- Norway maple
- Red maple, black maple
- Red oak, swamp white oak, hills oak
- Linden (American and littleleaf)
- Kentucky coffeetree
- Bald cypress
- Amur maple
- Crabapple (several varieties)
- Japanese tree lilac
- American plum
- Bradford callery pear (Chanticleer pear)
- Hawthorn (several varieties; thornless varieties may be desirable in some situations)
- Amur maple

*Shrubs*

- Redosier dogwood
- Gray dogwood
- Viburnum (several varieties)
- Clavey honeysuckle, Arnold's red honeysuckle
- Sumac (fragrant, smooth, and staghorn)
- American plum
- Van Houtte spirea
- Amelanchier

- Lilac
- Potentilla

*Evergreens*

- Black Hills spruce
- Juniper (several varieties)
- Austrian pine

*Vines*

- Virginia Creeper

As in the grasses, native plants seem to better adapt to the roadside environment, particularly because they are already acclimated to midwest soils, rainfall, drought, and other environmental factors. Based on research done by others, IDOT is moving away from using any soil amendments; it is using larger mulched areas, and, because of highly compacted, impervious soils, it is planting most plants higher than they were grown in the field to facilitate root establishment.

The native plants selected are mixed with nonnative plants that have proven hardy, in much the same way that warm- and cool-season grasses are blended, giving the landscape a broader-based tolerance to environmental factors.

Designs based on planned management can have a wide range of total management levels, from zero maintenance to intensive management. The level of management of a public right-of-way ideally approaches zero, requiring little input of chemicals, fertilizer, or manpower. With the goal of nearly zero management, the selection of plant material, whether it is turf grasses, wildflowers, or woody plants, should be a carefully conceived process using proven native plant material that will thrive with little care after planting.

*Publication of this paper sponsored by Committee on Roadside Maintenance.*

# Identification, Preservation, and Management of Minnesota Roadside Prairie Communities

KATHRYN E. BOLIN, NANCY J. ALBRECHT, AND ROBERT L. JACOBSON

Surveys to identify stands of high-quality native vegetation were initiated by the Minnesota Department of Transportation on highway and railroad rights-of-way in 1988. They were based on two earlier right-of-way surveys (in 1978 and 1980) that had identified 25 corridors supporting high- to relatively good-quality native prairie vegetation in Minnesota. The recent surveys indicate that 30 to 50 percent of the corridors identified in 1978 and 1980 have been lost because of railway abandonment, reconveyance of land to adjacent landowners, and highway reconstruction and maintenance activities. This finding is consistent with the overall dramatic decline of tallgrass prairie on rights-of-way throughout its range in Minnesota and the rest of the upper Midwest. In order to prevent further degradation of these valuable roadside prairie communities, Minnesota's Departments of Natural Resources and Transportation initiated an innovative and cooperative roadside prairie preservation and management program. The program is based on a preservation and management strategy that had been in use on Minnesota Trunk Highway 56 (TH 56) since 1983, where high-quality prairie vegetation was identified by early survey work. Since that time, the TH 56 right-of-way has been maintained using prescribed burning as the primary management tool, while at the same time mowing and herbicide use have been minimized. Management of TH 56 to enhance the right-of-way prairie there has been so stressful that this corridor was designated as Minnesota's first wildflower route on August 19, 1989.

Rights-of-way have been recognized historically as refuges for native vegetation communities (*1*). This is particularly true in the tallgrass prairie regions along highway and railroad rights-of-way constructed in the late 1800s and early 1900s. Before European settlement, the dominant vegetation community of nearly the entire upper Midwest was tallgrass prairie. However, since that time, large expanses of native vegetation (including prairie) have become increasingly less common. Nearly one-third of Minnesota was once native prairie (Figure 1). Now less than 1 percent of it is left (*2*). Today, much of the native prairie that remains is found in areas that were unsuitable for agriculture, such as cemeteries, steep bluffs, and along rights-of-way.

When the railroads were first built in the 1800s and 1900s, they transected the virgin tallgrass prairie of the upper Midwest. Prairie species reestablished back into the railroad rights-of-way that were initially disturbed by construction. Subsequently, many highways followed these early transportation routes and were built adjacent to railroad tracks. Frequently, long narrow corridors of prairie were isolated and protected in the shared highway and railroad rights-of-way. These corridors were left undisturbed by agriculture, whereas most of the rest of the surrounding prairie disappeared. Periodic fires along railroad rights-of-way have enabled the fire-adapted prairie species to maintain a foothold there (*4*).

There are a number of benefits to working with native vegetation along roadsides. Practical benefits to highway departments include the potential for a reduction in the cost of roadside maintenance and increased erosion control when native vegetation communities are present. Ecological benefits include the preservation of habitat for wildlife that uses roadsides for nesting cover and forage (*5,6*), protection of rare plant and animal species, potential preservation of natural genetic exchange between these species along linear corridors (*7*), and the protection of a significant percentage of the remaining tallgrass prairie communities in the upper Midwest. Rights-of-way containing native vegetation also serve as a seed source for future restoration efforts. Finally, native plants provide a display of seasonal color changes along roadsides, a natural beautification. Current state-of-the-art techniques for restoring prairie and other native vegetation communities are costly because high-quality native seed is often in short supply. Because the establishment characteristics of native species from seed is not the same as more traditional turf mixes used by many highway departments, restorations on roadsides are often deemed unsuccessful (especially in the short term), whereas management of existing native vegetation communities provides more immediate results for state agencies and the public to see.

In 1978, The Nature Conservancy (TNC) and the Minnesota Department of Natural Resources (MnDNR) sponsored a vegetation survey of highway and railroad rights-of-way in central and western Minnesota. Results of this survey published in 1983 (*8*) identified 16 corridors (544 mi) that contained high- to fair-quality native prairie species. In 1980, Bolin et al. (unpublished) identified nine additional corridors in southeastern Minnesota that contained high-quality native prairie vegetation. Beginning in 1988, additional roadside survey work has been performed by the Minnesota Department of Transportation (Mn/DOT).

The current survey work was initiated as the first step in part of the development of a comprehensive vegetation management plan for Minnesota roadsides. It was found that the locations of high-quality remnants of native vegetation on rights-of-way, if known, were not easily accessed by state

K. E. Bolin, Minnesota Department of Natural Resources, Region V, P.O. Box 6247, Rochester, Minn. 55903. N. J. Albrecht, Minnesota Department of Natural Resources, Metro Region, 1200 Warner Rd., St. Paul, Minn. 55106. R. L. Jacobson, Minnesota Department of Transportation, 1400 Gervais Ave., Maplewood, Minn. 55109.

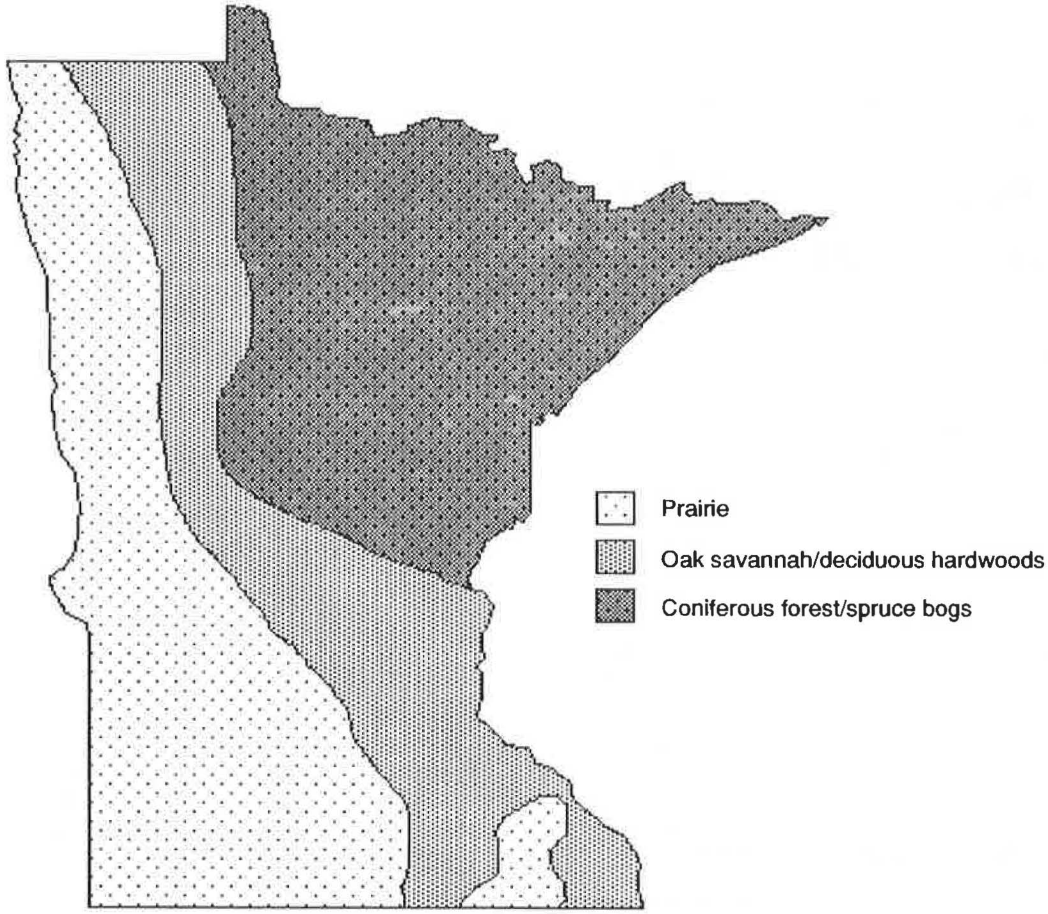



**FIGURE 1 Major vegetation communities of Minnesota, circa 1850s (*3*).**

agencies, private corporations (railroads), or the public. As a result, native plant communities were not addressed when highways, railroads, and their associated rights-of-way were upgraded or managed. Traditional management practices had concentrated on weed control by using herbicides and mowing as opposed to native plant community enhancement through periodic prescribed burns. It was hypothesized that only through a combination of the two approaches could a state-wide vegetation management program on roadsides be successful.

The results of the surveys that began in 1988 and of a cooperative prairie corridor preservation and management strategy implemented in 1983 along Minnesota Trunk Highway TH 56 in southeastern Minnesota by MnDNR and Mn/DOT are described. Information gained from these investigations provides the background that will be instrumental in the formation of a state-wide comprehensive roadside vegetation management plan for the state of Minnesota.

## MATERIALS AND METHODS

### Background Research

Several criteria were used to direct the surveyors to roadsides potentially having high-quality native vegetation (prairie) still existing in the right-of-way. These were (a) a computer search of the MnDNR Natural Heritage Program data base for rare plant elements on rights-of-way; (b) corridors identified by earlier surveys (*8* and Bolin et al., unpublished); (c) a questionnaire sent out to knowledgeable individuals representing various state agencies, universities, and conservation groups; and (d) trunk highways that had either active or recently abandoned railway lines adjacent to them.

### Vegetation Surveys

All previously identified corridors were resurveyed using a modified version of the method of Borowske and Heitlinger (*8*). Nineteen additional highways were surveyed using the same method. The presence of five or more prairie species served as an indicator of high-quality (A) prairie vegetation. Disturbed, or fair-quality prairie vegetation, was designated B and was characterized by having fewer than five prairie species present or having a large number of nonnative species present. Quality C indicated no prairie species were present. Highways were reviewed in the field from a moving vehicle for indicator species. Stops were made periodically when indicator species were observed and high-quality assessments were made. Inventories were performed at sites that were determined to be Quality A. Additional notes were taken relating the quality of the entire corridor to the sites surveyed.

### Species Inventory

For Quality A sites that were surveyed, a species inventory was conducted along a stretch of roadside that was approximately 100 ft long. The inslope, ditch bottom, backslope, and the railroad right-of-way were all included in the survey. All plants that were blooming and any identifiable species still in the vegetative stage were documented using a simplified version of Braun-Blanquet's floristic system (*9*). This system identified the species present along with their relative cover, abundance, and sociability (growth habit, e.g., growing singly, in groups, or as a mat). The following additional information was also frequently recorded: site conditions (dry, mesic, or wet); length and continuity of the native vegetation stand; adjacent land use; indications of disturbance by railroad, utility, or highway maintenance activities; presence of natural or constructed fire breaks (for future burn management); potential for extending the stand by restoration; and potential for harvesting seed from the site for future restoration purposes.

### Preservation and Management of TH 56 Prairie Communities

TH 56 was initially identified as containing a high diversity of prairie species in 1980 by Bolin et al. Since that time, it has been managed cooperatively by Mn/DOT and MnDNR. In 1983, TH 56 was posted with Do Not Mow and Do Not Spray signs so that maintenance personnel, utilities, and adjacent landowners did not mow, hay, or spray herbicide in the right-of-way. Permits allowing utility construction and herbicide use in the right-of-way have been closely monitored and have sometimes been restricted by the Mn/DOT area maintenance engineer when it was deemed that their activities posed a threat to the prairie communities present. In addition, TH 56 has recently been posted with special Wildflower Route signs to notify the public that the highway is of special significance.

Also since 1983, parts of the TH 56 right-of-way have been managed using prescribed burns performed by crews composed of Mn/DOT and MnDNR personnel. A prescribed burn proposal was developed for each segment to be burned, indicating the size of the segment (usually between 0.25 and 1.0 mi as determined by natural or constructed fire breaks); fire prescription, including season, relative humidity, temperature, and wind direction; and purpose of the burn, e.g., for brush control or prairie vegetation enhancement. Proposals were approved by the area MnDNR forester and the Pollution Control Agency. Local enforcement agents, fire departments, and adjacent landowners were also contacted. If the right-of-way was shared with a railroad or private utility company, its permission was also obtained to perform the burn.

During a burn, traffic was monitored by Mn/DOT personnel at all times, and the work areas were marked with Roadwork Ahead signs. If weather conditions fit those prescribed in the burn proposal, traffic was allowed to flow as usual. However, if wind direction shifted affecting visibility, then traffic was shut down to one lane, or a pilot car was used to guide traffic through the work area. If traffic flow was stopped or delayed, waiting vehicle drivers were given information packets describing what was being accomplished with the prescribed burns.

Prescribed burns have been designed to accomplish several objectives: (a) stop the encroachment of brush into the prairie remnant; (b) decrease the abundance of cool-season, nonnative grasses that are invariably present; and (c) enhance the growth of native warm-season grasses and herbaceous species (forbs) that are present. In order to accomplish these objectives, burns are performed in late April to mid-May, when the cool-season grasses are actively growing and the native warm-season grasses are still dormant. Individual segments have been burned on a 3- to 5-year cycle. Qualitative assessments of the burn are performed later in the season and again the following year.

## RESULTS

The current vegetation surveys reviewed 35 Minnesota trunk highways (approximately 3,000 mi) for remaining stands of native vegetation communities (Table 1). Included in the current surveys were the trunk highways surveyed by Borowske and Heitlinger in 1978 (*8*) and those surveyed by Bolin et al. in 1980 (unpublished). It is estimated that 30 to 50 percent of the Quality A rights-of-way identified in 1978 and 1980 have either been destroyed or have degraded significantly since that time (see Figure 2 and Table 2). The major causes for loss or degradation of native vegetation communities in rights-of-way are (a) abandonment of the rail line by the railroad and subsequent reconveyance of the railroad right-of-way to adjacent landowners, who put the land to agricultural use; (b) highway reconstruction, repair, and maintenance activities; (c) utility and railroad maintenance activities; and (d) haying or mowing of rights-of-way by adjacent landowners. Approximately 15 percent (450 mi) of the total miles of trunk highways reviewed since 1988 contain Quality A native vegetation. Few of the corridors had long continuous stretches of high-quality native vegetation. However, many corridors contain longer intermittent stretches of high-quality prairie vegetation.

Rights-of-way that were considered to be Quality A on initial review were nearly always found to contain a very high diversity of native species. Frequently, 40 to 50 native species were identified when an inventory was conducted on Quality A rights-of-way. Native grasses served as the best criteria for gauging the quality of right-of-way vegetation. However, the presence of rare species also served as an excellent indicator of high-quality right-of-way. Many native forbs appeared to be able to persist in areas dominated by disturbance indicator species such as smooth brome (*Bromus inermis*). Quality B right-of-way vegetation was frequently found to be dominated by *Bromus inermis*, with a few native forbs also being present. Table 3 presents some of the species found in Minnesota rights-of-way.

All surveyed rights-of-way were found to contain vegetation communities that were in a state of flux. Native species were found intermingled with nonnative species that had either been planted on the inslopes of the road bed or that had invaded the right-of-way from adjacent fields. The highest-quality areas were generally found in the ditch bottom, backslope, and railroad right-of-way (when there was a railroad present). It is hypothesized that the fire history of these areas has played a major role in the ability of the native vegetation communities to resist invasion by introduced cool-season species and noxious weeds; or, in the case of prairie, encroachment by woody species.

TABLE 1 COMMUNITY TYPE AND QUALITY OF RIGHTS-OF-WAY SURVEYED IN 1988 AND 1989

| Highway | Community Type | Quality |
|---|---|---|
| T.H. 1 | P | A |
| T.H. 2* | NCF | B |
| T.H. 4 | P | B |
| T.H. 6 | NCF | B |
| T.H. 7 | P | A (Intermittent) |
| T.H. 9* | P | A |
| T.H. 10* | P | A (Intermittent) |
| T.H. 11 | P/DH/NCF | A |
| T.H. 13* | P | C |
| T.H. 14 | P | A (Intermittent) |
| T.H. 22* | P | A (Intermittent) |
| T.H. 23* | P | B |
| T.H. 30 | P | B |
| T.H. 32* | P | C |
| T.H. 34 | NCF | B |
| T.H. 46 | NCF | B |
| T.H. 52 (old)* | P | B |
| T.H. 53 | NCF | B |
| T.H. 56* | P | A |
| T.H. 59 | P | A (Intermittent) |
| T.H. 60 | P | A |
| T.H. 61* | NCF | B |
| T.H. 65* | P | B |
| T.H. 71* | NCF | B |
| T.H. 72 | NCF - bogs | B |
| T.H. 73 | NCF | B |
| T.H. 75* | P | A (Intermittent) |
| T.H. 83 | P | B |
| T.H. 89 | NCF/DH | B |
| T.H. 102* | P | C |
| T.H. 169 | NCF | C |
| T.H. 210 | P/DH/NCF | B |
| T.H. 212 | P | A (Intermittent) |
| T.H. 218* | P | A |
| T.H. 371* | P/DH/NCF | A (Intermittent) |

P = Prairie
DH = Deciduous Hardwood Forest
NCF = Northern Coniferous Forest

*Previously Surveyed

The TH 56 right-of-way prairie community that was identified in 1980 by Bolin et al. (unpublished) has flourished since the prescribed burn management plan was implemented by Mn/DOT and MnDNR in 1983. Fire management has begun to decrease the encroachment of brush into the right-of-way and has stimulated the comeback of native prairie species. There are a number of rare plants that can be found in rights-of-way (Table 3). Populations of two rare species found along TH 56, rattlesnake master (*Eryngium yuccifolium*) and wild quinine (*Parthenium integrifolium*), have maintained or increased slightly in areas that have been managed.

## DISCUSSION

The loss or degradation of 30 to 50 percent of the native vegetation corridors that were identified in 1978 and 1980, coupled with the overall decline of tallgrass prairie communities in Minnesota, has prompted state agencies and conservation groups to work together in identifying, preserving, and managing what remains of these communities in rights-of-way. The methodology (using native grasses as indicator species) for identifying high-quality native vegetation communities in rights-of-way is relatively simple, but it does require botanical or ecological expertise. This method appears to be reliable as a way to cover many miles of right-of-way in a relatively short period of time, leaving more detailed inventories for later. The preservation and management of prairie communities in rights-of-way requires the cooperation of all of the entities that use or share the right-of-way. The unique skills of personnel both from highway departments and departments of natural resources are required to safely and effectively manage right-of-way native vegetation communities.

Roadside rights-of-way are environments that are continually disturbed and exposed to harsh conditions; whether it be from human activities such as automobile exhaust, salt, and applied chemicals, or from naturally occurring adverse climactic conditions like drought, flooding, and extreme temperatures. It was apparent from the current surveys that rights-of-way containing established native vegetation communities were less susceptible to drought kill, weedy invasion, and erosion than those rights-of-way that contained introduced

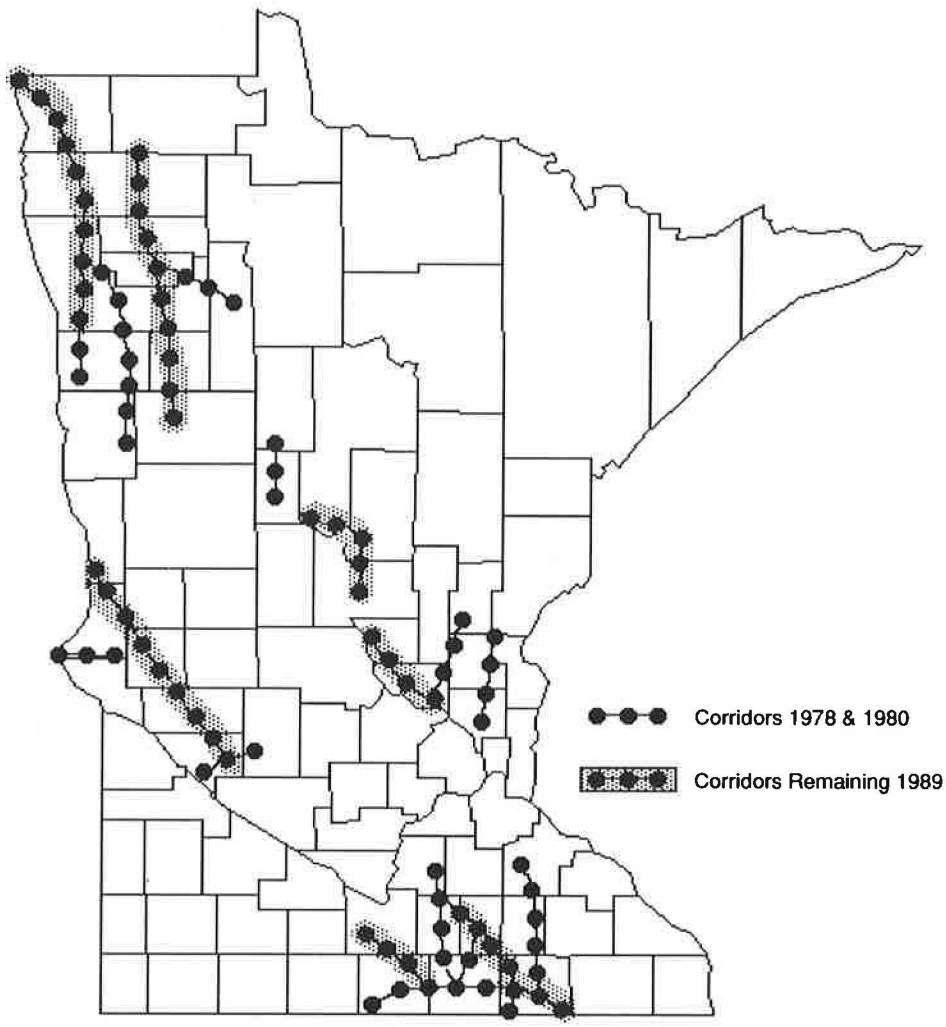



**FIGURE 2 Prairie corridors lost since 1980.**

species. It is thought that this is because native plants are better adapted to survive in Minnesota than nonnative plants, and diverse plant communities (such as a prairie community) are more able to withstand adverse conditions and frequent disturbance than monoculture or low-diversity plantings composed of nonnative grasses or forbs.

With the management plan that has been implemented on the TH 56 right-of-way, it has been found that the use of herbicides and mowing for weed control have decreased. Fire has reduced chemical and mechanical removal of brush as well. This translates into direct cost savings over the years as herbicide use and mowing decrease. It is anticipated that under the present management plan that emphasizes the enhancement of the native prairie community along TH 56, the need for mowing will decrease to keeping the inslopes mowed short, to keeping the sight lines clear for safety purposes, and to patch-mowing of weeds. This type of management plan decreases disturbance of the right-of-way by human activities substantially. Remaining disturbances are then left to natural causes (which are beyond control), but for which the native species are adapted to survive.

In order to further explore the possibilities and benefits of working with native vegetation on Minnesota roadsides, the state has formed a task force composed of representatives from various state agencies, the University of Minnesota, private corporations, and the public sector. The task force serves to make recommendations and suggest guidelines for various state programs. The task force is also able to serve as an interface between the state and the public, with its growing interest in the use of native wildflowers and grasses for highway beautification. In addition, Mn/DOT and MnDNR have begun cooperation, on a state-wide basis, in developing an integrated roadside vegetation management program. An interagency committee has been formed to set guidelines for such a program. Management to enhance native prairie communities similar to that being used along TH 56 will begin along a number of Minnesota highways in 1990 (Figure 3). Because these prairie communities are composed of native grasses and wildflowers, enhancement of these areas has enabled Mn/DOT to designate them as wildflower routes. It is felt that a combination of management to enhance existing prairie and restoration (usually locally occurring native species) to expand existing stands along these corridors, will create a scenic wildflower route system that is both aesthetically pleasing and requiring of low maintenance (Figure 3).

The designation of TH 56 as Minnesota's first wildflower route is testimony to the fact that the goals of Mn/DOT, MnDNR, and conservation groups can all be met, and at the

TABLE 2 CHANGE IN QUALITY OF CORRIDORS FROM 1978 TO PRESENT

| Highways Surveyed 1988-1989 | Surveyed 1978 | Surveyed 1980 | Δ Quality |
|---|---|---|---|
| T.H. 2 | yes | no | Decline |
| T.H. 9 | yes | no | No Change |
| T.H. 10 | yes | no | Decline |
| T.H. 13 | no | yes | Large Decline |
| T.H. 22 | no | yes | No Change |
| T.H. 23 | yes | no | Decline |
| T.H. 32 | yes | no | Large Decline |
| T.H. 52/56 | no | yes | Large Decline |
| T.H. 56 (Mower Co.)* | no | yes | Improved |
| T.H. 61 | yes | no | No Change |
| T.H. 65 | no | yes | No Change |
| T.H. 71 | yes | no | Decline |
| T.H. 75 | yes | no | Decline |
| T.H. 102 | yes | no | Large Decline |
| T.H. 218 | no | yes | No Change |
| T.H. 371 | yes | no | No Change |

*On managed segments

TABLE 3 SOME SPECIES OF SIGNIFICANCE FOUND IN PRAIRIE RIGHTS-OF-WAY

**Native Grasses - Prairie Indicators**

Big Bluestem (*Andropogon gerardi*)
Sideoats Grama (*Bouteloa curtipendula*)
Switch Grass (*Panicum virgatum*)
Little Bluestem (*Schizachyrium scoparium*)
Indian Grass (*Sorghastrum nutans*)
Prairie Cordgrass (*Spartina pectinata*)
Dropseeds (*Sporobolis* spp.)
Needle Grasses (*Stipa* spp.)

**Native Forbs - Prairie Indicators**

Lead Plant (*Amorpha* spp.)
Stiff Tickseed (*Coreopsis palmata*)
Common Oxeye (*Heliopsis helianthoides*)
Blazingstars (*Liatris* spp.)
Prairie Clovers (*Petalostemum* spp.)
Prairie Phlox (*Phlox pilosa*)
Coneflowers (*Ratibida* spp.)

**Rare Plant Species**

Small White Ladyslipper*
(*Cypripedium candidum*)
Rattlesnake Master*
(*Eryngium yuccifolium*)
Prairie Bush Clover**
(*Lespedeza leptostachya*)
Wild Quinine*
(*Parthenium integrifolium*)
Western Prairie Fringed Orchid**
(*Platanthera praeclara*)

**Disturbance Indicator Species**

Quack Grass (*Agropyron repens*)
Smooth Brome (*Bromus inermis*)
Canada Thistle (*Cirsium canadensis*)
Leafy Spurge (*Euphorbia esula*)
Sweet Clover (*Melilotus officinalis*)
Kentucky Bluegrass (*Poa pratensis*)
Timothy (*Phleum pratense*)

*Protected by the State of Minnesota
**Protected by the Federal Government

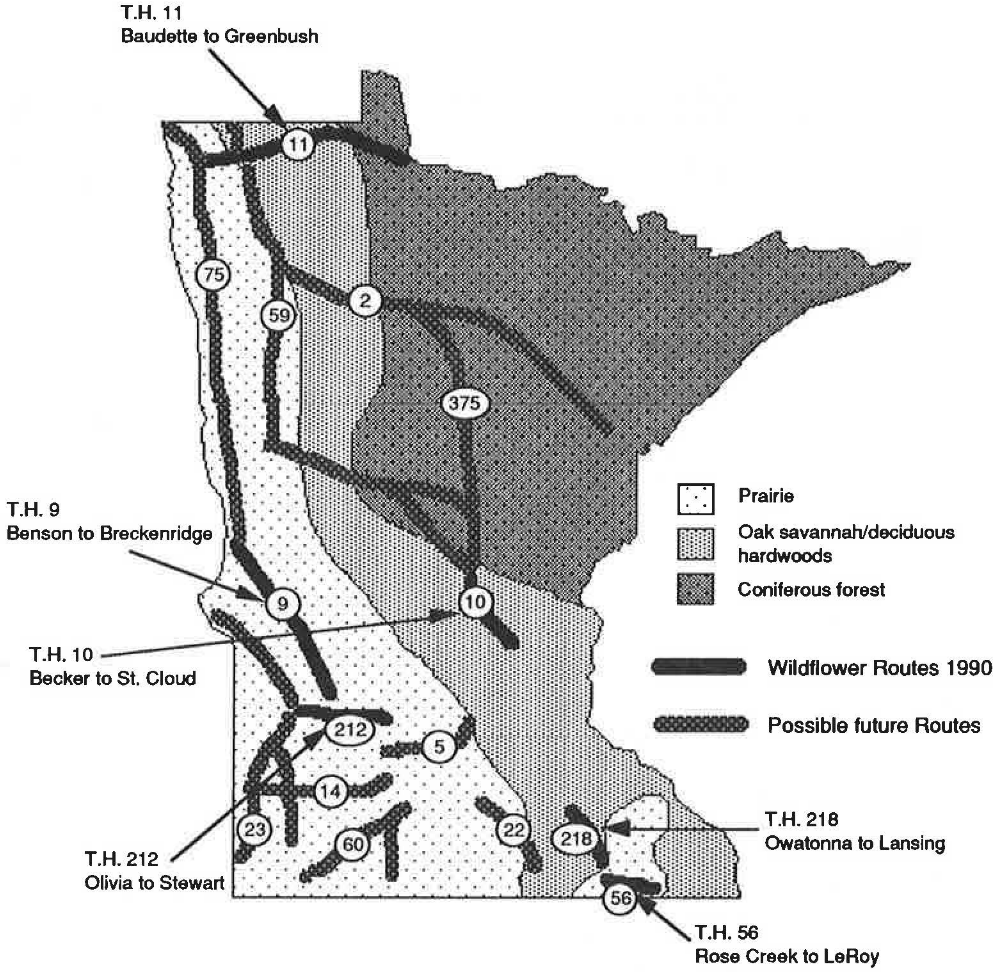



**FIGURE 3 Minnesota wildflower routes and high-quality corridors.**

same time the traveling public benefits by seeing part of Minnesota's natural heritage flourishing once again along Minnesota's roadsides.

## ACKNOWLEDGMENT

The authors would like to acknowledge the efforts of Mn/DOT, District 6B, Maintenance (Owatonna, Minnesota), in the preservation and management of the prairie vegetation along TH 56 in southeastern Minnesota.

## REFERENCES

1. B. Shimek. The Persistence of the Prairie. *University of Iowa Studies in Natural History*, Vol. 11, 1925, pp. 3–24.
2. K. Wendt. *Mesic Prairie Element Abstract.* Minnesota Department of Natural Resources, Minneapolis, 1987.
3. J. B. Moyle and E. W. Moyle. *Northland Wildflowers.* University of Minnesota Press, Minneapolis, 1977, 236 pp.
4. J. White. Why Bother to Protect Prairies Along Railroads? *Proc., 9th North American Prairie Conference*, July 29 to Aug. 1, 1984. G. K. Clambey and R. H. Pemble, eds., Moorhead, Minn., 1986.
5. *Highway-Wildlife Relationships, Vol. 1.* Report FHWA-RO-76-4. FHWA, U.S. Department of Transportation, 1975, 183 pp.
6. *Highway-Wildlife Relationships, Vol. 2.* Report FHWA-RO-76-5. FHWA, U.S. Department of Transportation, 1975, 417 pp.
7. D. M. Reed and J. A. Schwarzmeier. The Prairie Corridor Concept: Possibilities for Planning Large-Scale Preservation and Restoration. *Proc., 5th Midwest Prairie Conference*, D. C. Glenn-Lewin and R. Q. Landers, Jr., eds., Iowa State University, Ames, 1978, pp. 158–165.
8. J. R. Borowske and M. E. Heitlinger. A Survey of Native Prairie on Railroad Rights-of-Way. *Proc., 7th North American Prairie Conference*, C. L. Kucera and J. Bielefeldt, eds., Aug. 4–6, 1980, Springfield, Mo., 1983, pp. 39–44.
9. J. Braun-Blanquet. *Plant Sociology: The Study of Plant Communities*, D. D. Fuller and H. S. Conrad, eds., Hafner, London, 1965, 439 pp.

*Publication of this paper sponsored by Committee on Roadside Maintenance.*

# Pennsylvania State University–PennDOT Roadside Research Project: Wildflower Evaluation

G. T. LYMAN AND A. E. GOVER

As part of a cooperative project between Pennsylvania State University and the Pennsylvania Department of Transportation, a study was initiated to investigate the performance of 50 different wildflower species for use on Pennsylvania's roadsides. Each species was evaluated for percent cover, percent weed invasion, and percent of the plot covered by blooms. The wildflowers were planted in the spring of 1988; their performance is evaluated through 1989.

A cooperative research project between Pennsylvania State University and the Pennsylvania Department of Transportation (PennDOT) was initiated in October 1985. The purpose of this research was to investigate several aspects of roadside vegetation management including brush control, Canada thistle control in crownvetch, and plant growth regulator use for turf areas. In 1988, the scope of the project increased and included the investigation of wildflowers for use on Pennsylvania's roadsides. PennDOT roadside managers were interested in using wildflowers but needed information on species selection. In April 1988, 50 wildflower species were screened as part of a national test coordinated by Pure Seed Testing Inc. of Oregon. The objective of this test was to evaluate the performance of these species over several growing seasons. The most successful wildflower species were to be incorporated into mixes for use on Pennsylvania's roadsides.

## MATERIALS AND METHODS

The planting site is located near State College, Pennsylvania, on a Hagerstown silt loam soil that was previously used for alfalfa production. The alfalfa was eliminated with an application of Roundup (glyphosate) and 2,4–D on April 10, 1988. A total of 24 annuals and 26 perennial wildflower species were planted on April 20, 1988, in individual 5 × 5-ft plots and replicated three times. The seeding rates were predetermined by Pure Seed Testing Inc. A power take-off driven verticut unit from an Olathe overseeder was used to slit the soil approximately 0.5 in. deep on 3-in. centers. To simulate a drop spreader, the seed for each plot was suspended in 100 g of Milorganite and shaken on the plot using a quart mason jar with a perforated lid. A wind screen was placed around each plot during the seeding operation. In late October 1988, each plot was mowed and the residue was left in the plot. A list of the common names and scientific names of the wildflower species evaluated in 1988 is presented in Table 1.

The study site was not irrigated or fertilized during the growing season. The average rainfall for the study area is approximately 38 in./year. Adequate rainfall for germination fell during late April and early May of 1988. However, June, July, and most of August were extremely hot and dry. Because of this drought, total rainfall was lower than average for 1988. In the spring of 1989, higher than normal rainfall occurred. Rainfall accumulation data in inches for the growing seasons of 1988 and 1989 are as follows:

| | *1988 (in.)* | *1989 (in.)* |
|---|---|---|
| April | 1.5 | 0.70 |
| May | 4.20 | 6.15 |
| June | 0.92 | 8.80 |
| July | 3.35 | 5.47 |
| August | 5.88 | 0.55 |
| September | 2.97 | 3.17 |
| October | 1.27 | 3.50 |
| Growing season total | 20.09 | 28.34 |

The average rainfall for a growing season was 24.20 in.

Each plot was rated visually for percent of the plot area covered by the wildflower species, percent weed cover, and percent of the plot covered by blossoms. By the end of the 1988 season, the area of the plot not covered by the wildflowers was covered with weed growth. Only percent wildflower cover and percent blossom cover will be discussed. Ratings in 1988 were performed on June 13, June 27, July 12, August 9, and September 28. The 1989 ratings were performed on May 29, June 22, July 4, July 18, August 8, and September 28. The success of each species was determined by considering its competitiveness with weeds and its flowering characteristics. A wildflower was considered successful if it vegetatively covered at least 60 percent of the plot area and its flower production covered at least 30 percent of the plot area. Successful species will be considered as candidates for use in mixes. If a wildflower were being evaluated for a single species planting, 60 percent vegetative cover may not be considered adequate.

## RESULTS AND DISCUSSION

In general, the annual species established and produced cover sooner than the perennial species at the early rating periods in 1988. A total of 20 of the 24 annual species produced more than 60 percent vegetative cover in 1988 (Figure 1). These 20

Department of Agronomy, Pennsylvania State University, 119 Tyson Building, University Park, Pa. 16802.

TABLE 1 COMMON AND SCIENTIFIC NAMES FOR THE WILDFLOWER SPECIES EVALUATED IN 1988

| *Annual Species* | | *Perennial Species* | |
|---|---|---|---|
| *Scientific Name* | *Common Name* | *Scientific Name* | *Common Name* |
| Anagallis arvensis | pimpernel | Achillea millefolium | white yarrow |
| Centaurea cyanus dwf. | dwarf cornflower | Achillea millefolium rubra | red yarrow |
| Chrysanthemum coronarium | garland chrysanthemum | Anthemis tinctoria | chamomile |
| Clarkia amoena | farewell to spring | Aquilegia vulgaris | dwarf columbine |
| Clarkia unguiculata | clarkia | Cerastium biebersteinii | snow in summer |
| Coreopsis tinctoria | T. plains coreopsis | Cheiranthus allionii | Siberian wallflower |
| Cosmos bipinnatus | cosmos | Cheiranthus cheiri | English wallflower |
| Delphinium ajacis | rocket larkspur | Coreopsis lanceolata | lance-leaf coreopsis |
| Dimorphotheca aurantiaca | African daisy | Dianthus barbatus | sweet William |
| Eschscholzia californica | California poppy | Dianthus deltoides | maiden pinks |
| Gaillardia pulchella | Indian blanket | Echinacea purpurea | purple coneflower |
| Gilia capitata | globe gilia | Gaillardia aristata | blanketflower |
| Gypsophila elegans | baby's breath | Hesperis matronalis | dames rocket |
| Layia platyglosa | tidy tips | Ipomopsis rubra | standing cypress |
| Linanthus grandiflorus | mountain phlox | Linum perenne lewisii | blue flax |
| Linaria maroccana | spurred snapdragon | Myosotis sylvatica | forget-me-not |
| Linum grandiflorum rubrum | scarlet flax | Oenothera lanarkiana | evening primrose |
| Lobularia maritima | sweet alyssum | Oenothera missouriensi | Missouri primrose |
| Mertensia virginica | blue bells | Penstemon strictus | Rocky Mountain penstemon |
| Monarda citriodora | lemon mint | Ratibida columnifera | prairie coneflower |
| Nemophila menziesii | baby blue eyes | Rudbeckia hirta | black-eyed Susan |
| Papaver rhoeas | corn poppy | Sanguisorba minor | small burnet |
| Scabiosa stellata | scabiosa | Sanvitalia procumbens | creeping zinnia |
| Silene armeria | catchfly | Saponaria ocymoides | soapwort |
| | | Thymus serpyllum | wild thyme |
| | | Viola cornuta | Johnny jump up |

species are grouped on the basis of general growth patterns. The first group produced a flush of growth by June 5, and coverage remained stable throughout the season. Some examples of species in this group were garland chrysanthemum, tall plains coreopsis, and California poppy. A second group displayed a decline in cover after the initial flush of growth, and by the end of the season were typically invaded by weed growth. Examples of these species were farewell to spring, clarkia, spurred snapdragon, and corn poppy. A third group displayed a relatively slow establishment rate, yet coverage increased steadily throughout the season. Examples of species in this group are Indian blanket, cosmos, scarlet flux, and lemon mint.

All of the annual species bloomed during the 1988 season (Figure 2). The bloom production was diverse across the species and occurred throughout the season. Some species such as baby's breath, farewell to spring, and clarkia displayed profuse flower production for a short period of time. Others such as Indian blanket and catchfly flowered steadily throughout the season. Still others such as tall plains coreopsis, cosmos, and sweet alyssum started flowering later and peaked throughout August and September.

The growth and flowering of the perennials were slower than those of the annuals in 1988. A total of 14 out of the 26 species produced more than 60 percent coverage during 1988 (Figure 3). Coverage generally increased at a steady rate throughout the season with most species producing the best cover by the last rating date.

Ten perennials produced flowers during 1988 (Figure 4). Of these, only Siberian wallflower, black-eyed Susan, and blanketflower provided blossoms that covered more than 30 percent of the plot area. All other perennial species produced most vegetative growth.

Wildflower performance varied considerably in 1989 from that observed in 1988. In 1989, few annuals reseeded sufficiently for acceptable performance, whereas the growth of several perennials was impressive. Although 20 annual species were successful in 1988, only 3 provided more than 60 percent coverage in 1989 (Figure 5). The greatest percent coverage was achieved by rocket larkspur and tall plains coreopsis. These species provided excellent cover by June 22 and remained competitive throughout the season. Dwarf cornflower also provided excellent cover by June 22, but peaked in mid-July and was invaded by weeds at the end of the season. Five other species provided a peak in cover that approached 60 percent in early July, but were invaded by weeds at the end of the season. They were globe gilia, California poppy, sweet alyssum, catchfly, and mountain phlox. All other annual species provided less than 20 percent vegetative cover. The annuals with the best flower production were rocket larkspur, tall plains coreopsis, and dwarf cornflower (Figure 6). Peak flower production for the annuals occurred between June 22 and August 3.

Vegetative cover for the successful perennials tended to increase throughout the season. A total of 14 perennial species were successful in 1988, yet only 9 of these provided more than 60 percent vegetative cover in 1989 (Figure 7). They were blue flax, dames rocket, chamomile, red yarrow, white yarrow, prairie coneflower, lance-leaved coreopsis, black-eyed Susan, and evening primrose. The five species that performed well in 1988, but did not perform well in 1989, included blanketflower, English wallflower, standing cypress, Siberian wallflower, and small burnet. Sweet William and purple coneflower were considered unsuccessful in 1988, but were impressive in 1989.

The 11 species that provided over 60 percent coverage in

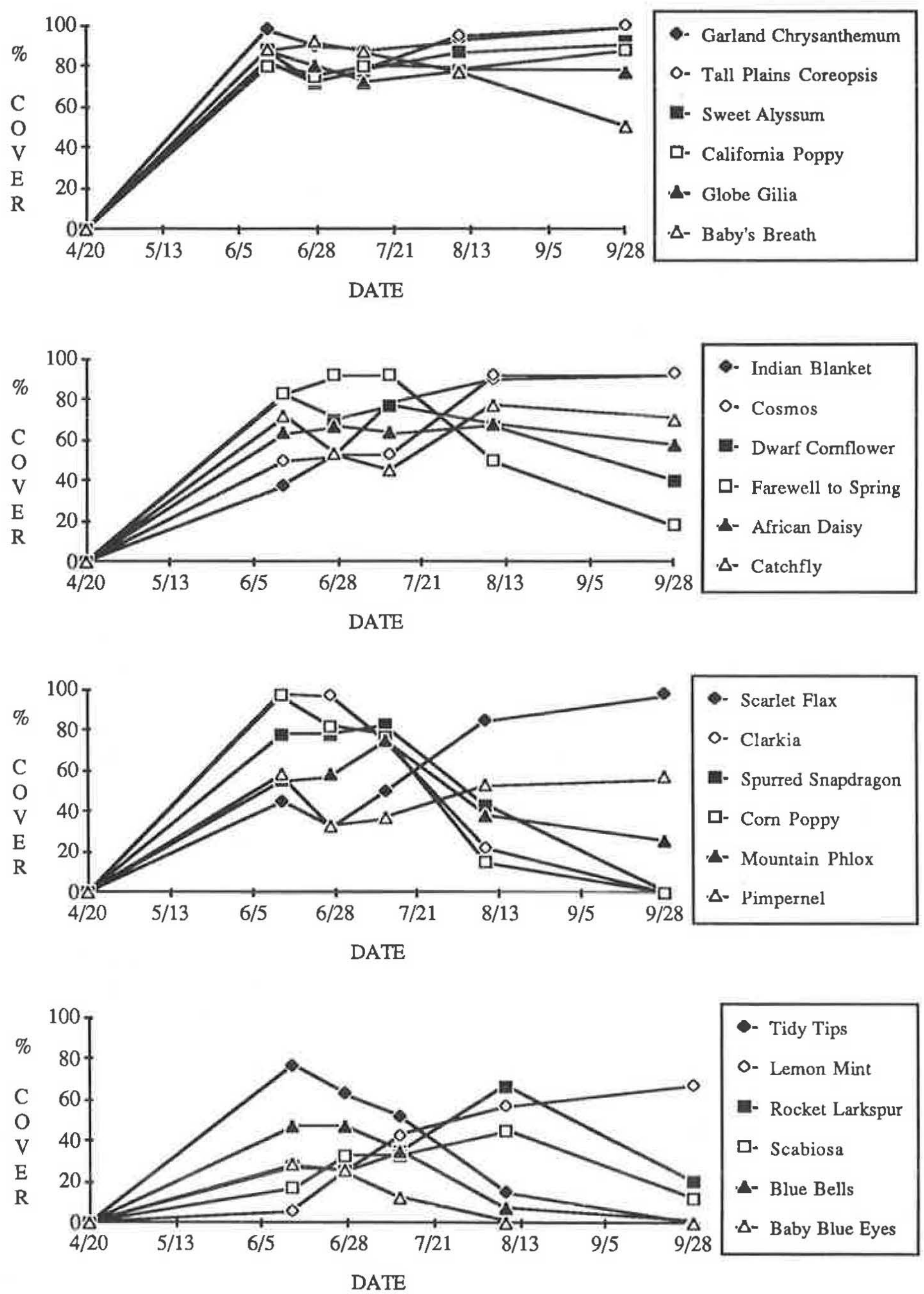



**FIGURE 1 Percent vegetative cover provided by annual wildflower species in 1988.**

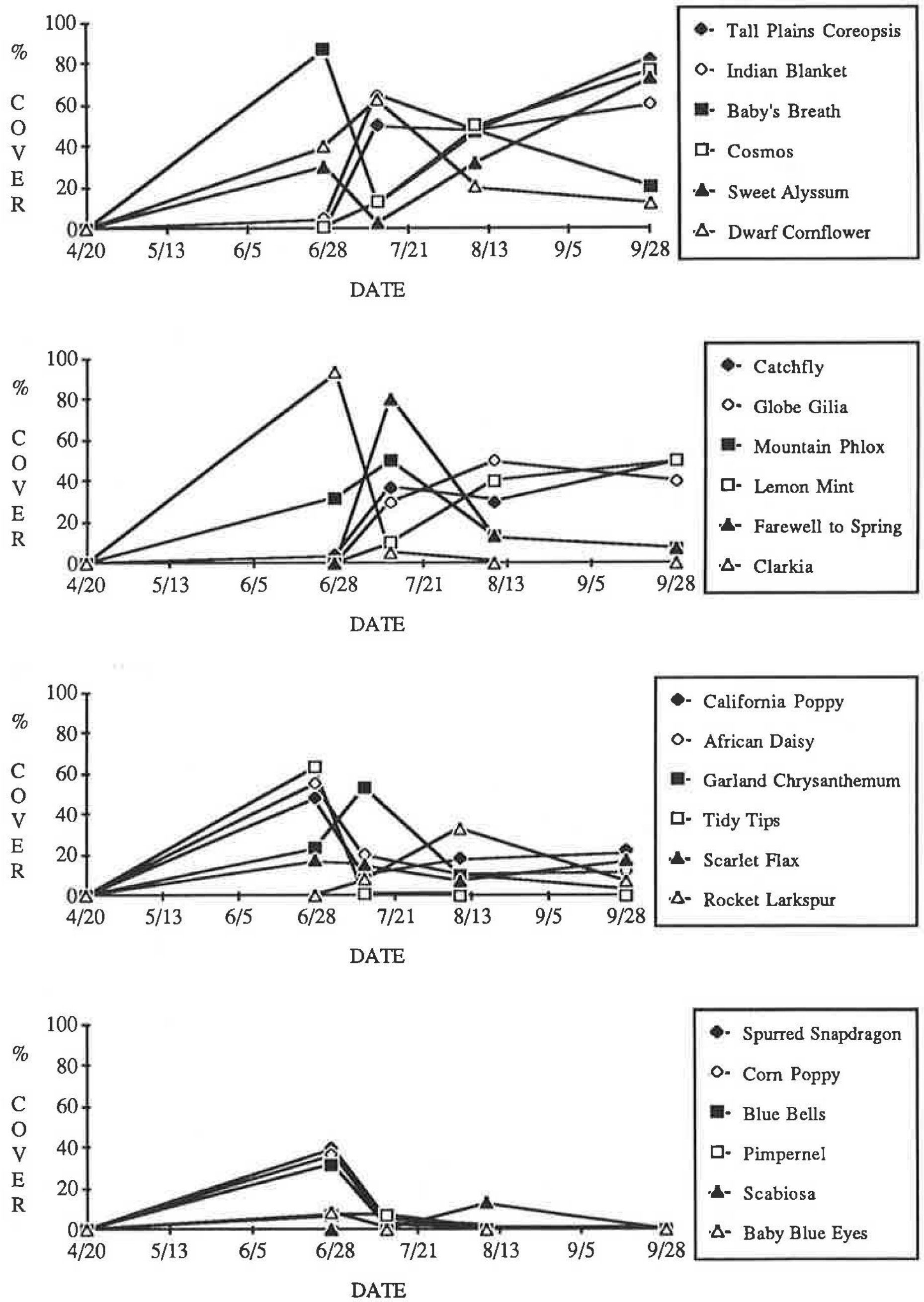



**FIGURE 2 Percent blossom cover provided by annual wildflower species in 1988.**

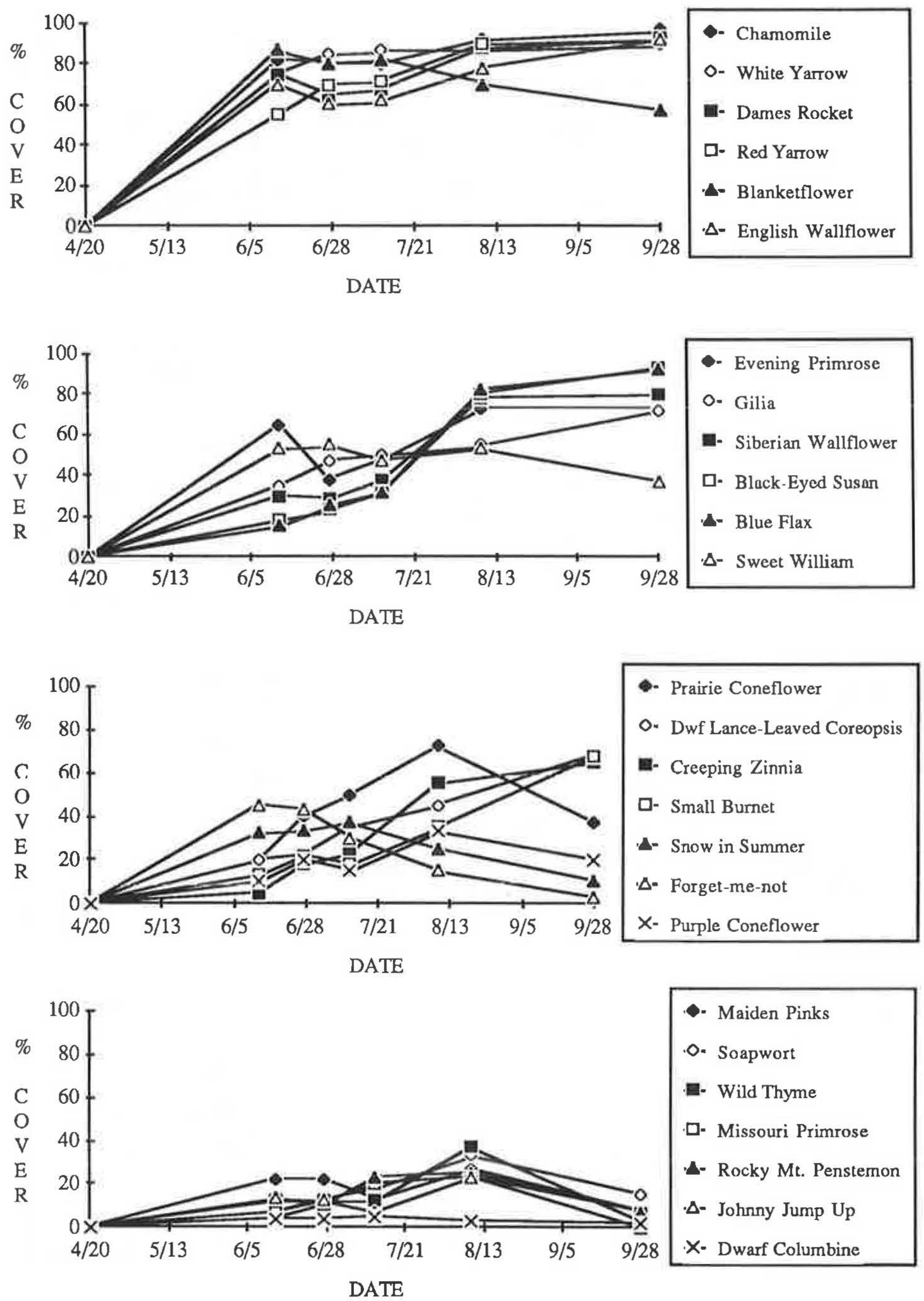



**FIGURE 3 Percent vegetative cover provided by perennial wildflower species in 1988.**

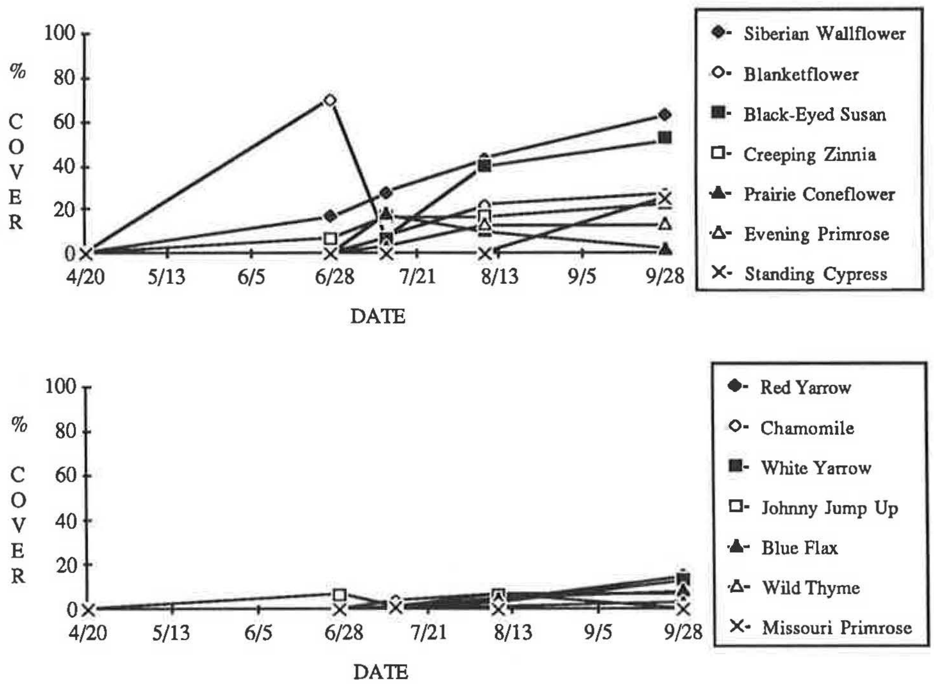



**FIGURE 4 Percent blossom cover provided by perennial wildflower species in 1988.**

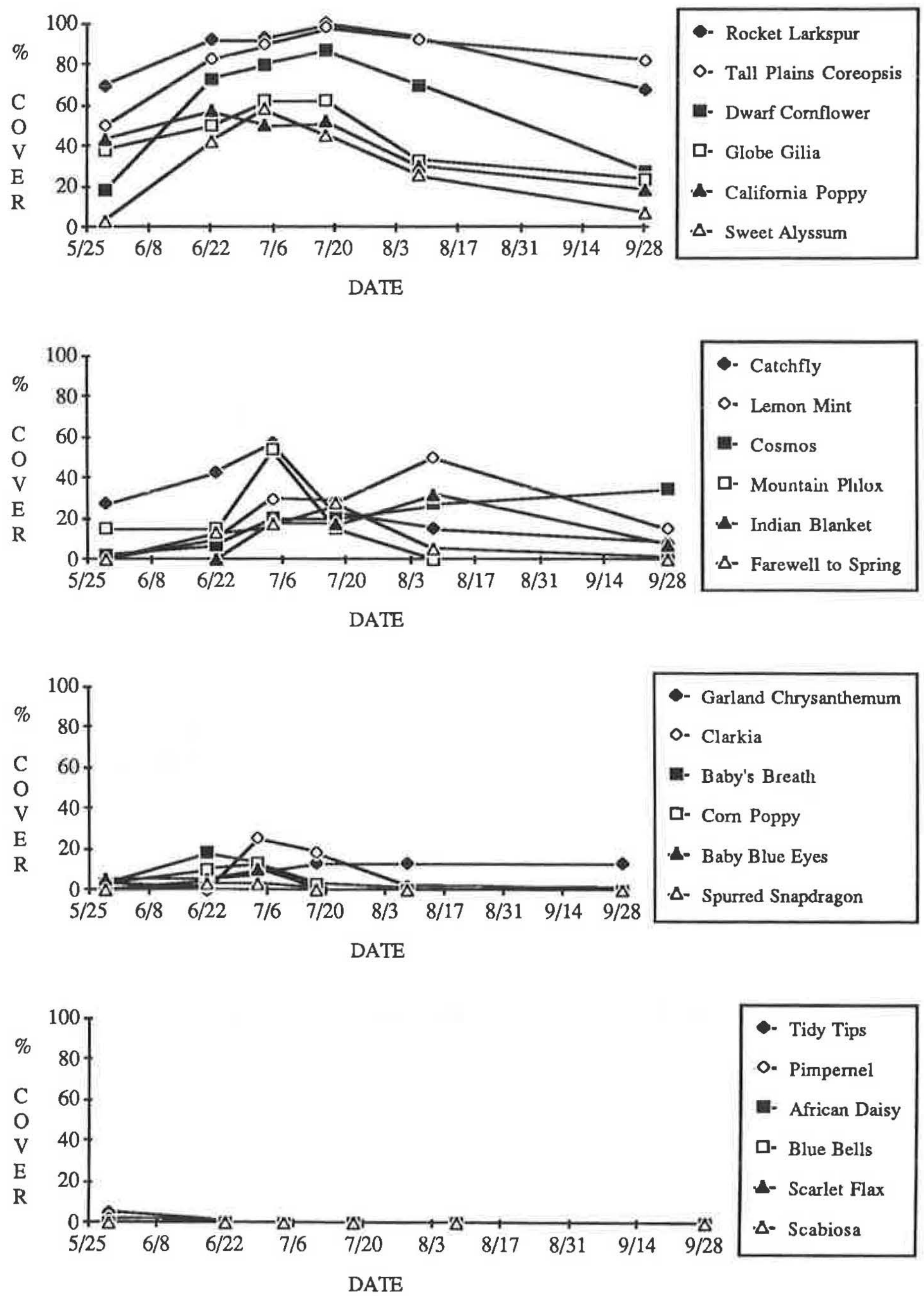



**FIGURE 5 Percent vegetative cover provided by annual wildflower species in 1989.**

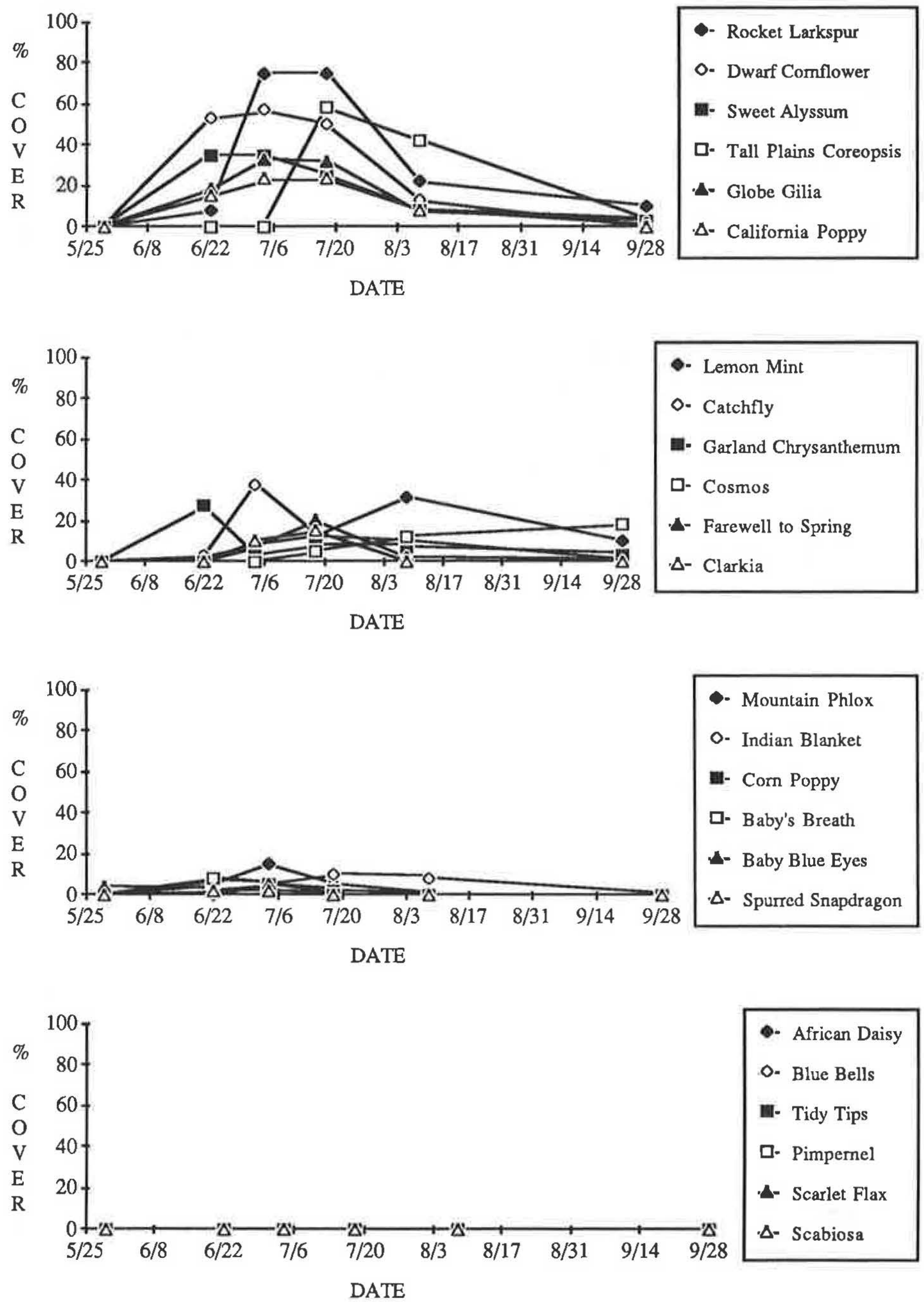



**FIGURE 6 Percent blossom cover provided by annual wildflower species in 1989.**

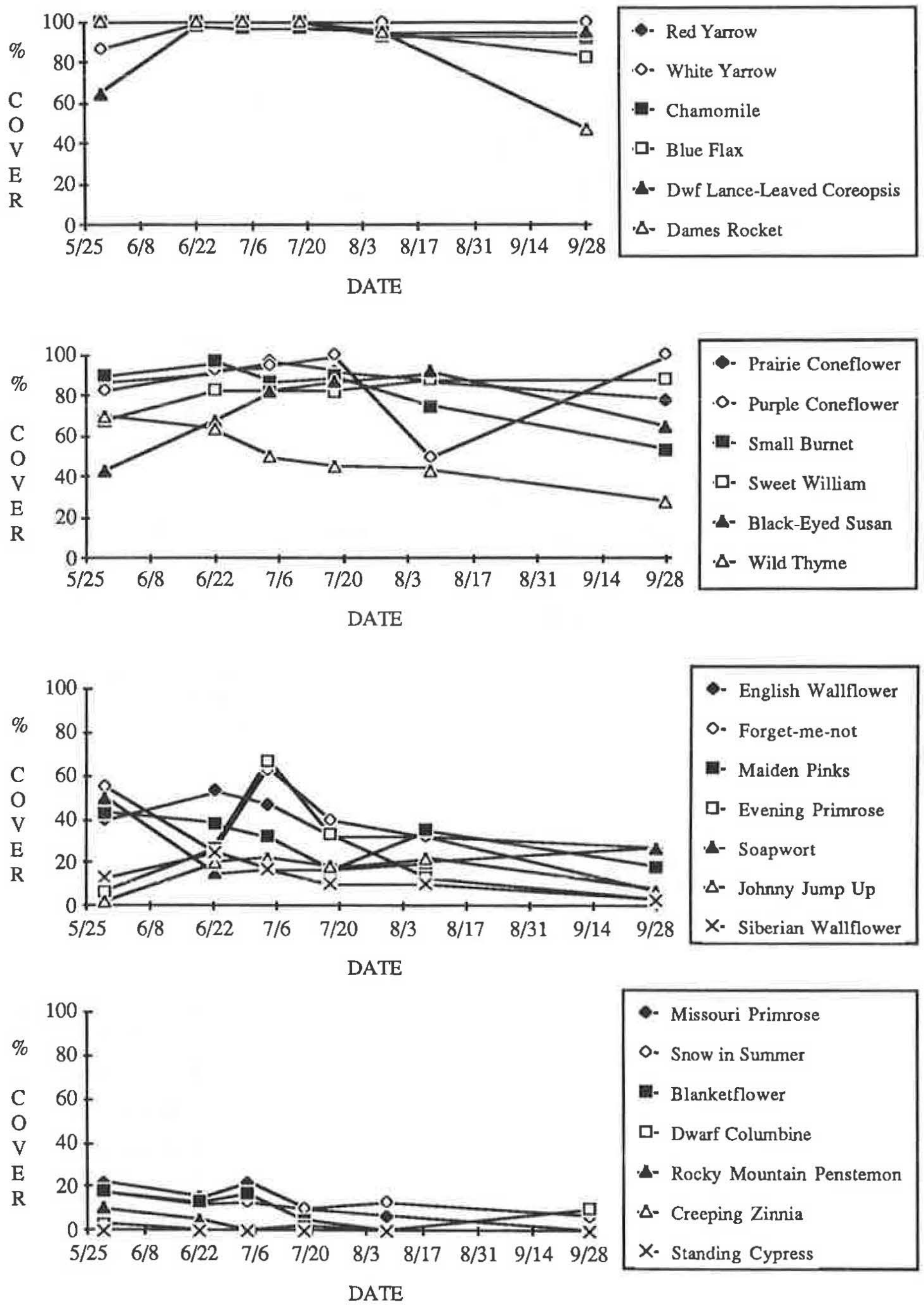



**FIGURE 7 Percent vegetative cover provided by perennial wildflower species in 1989.**

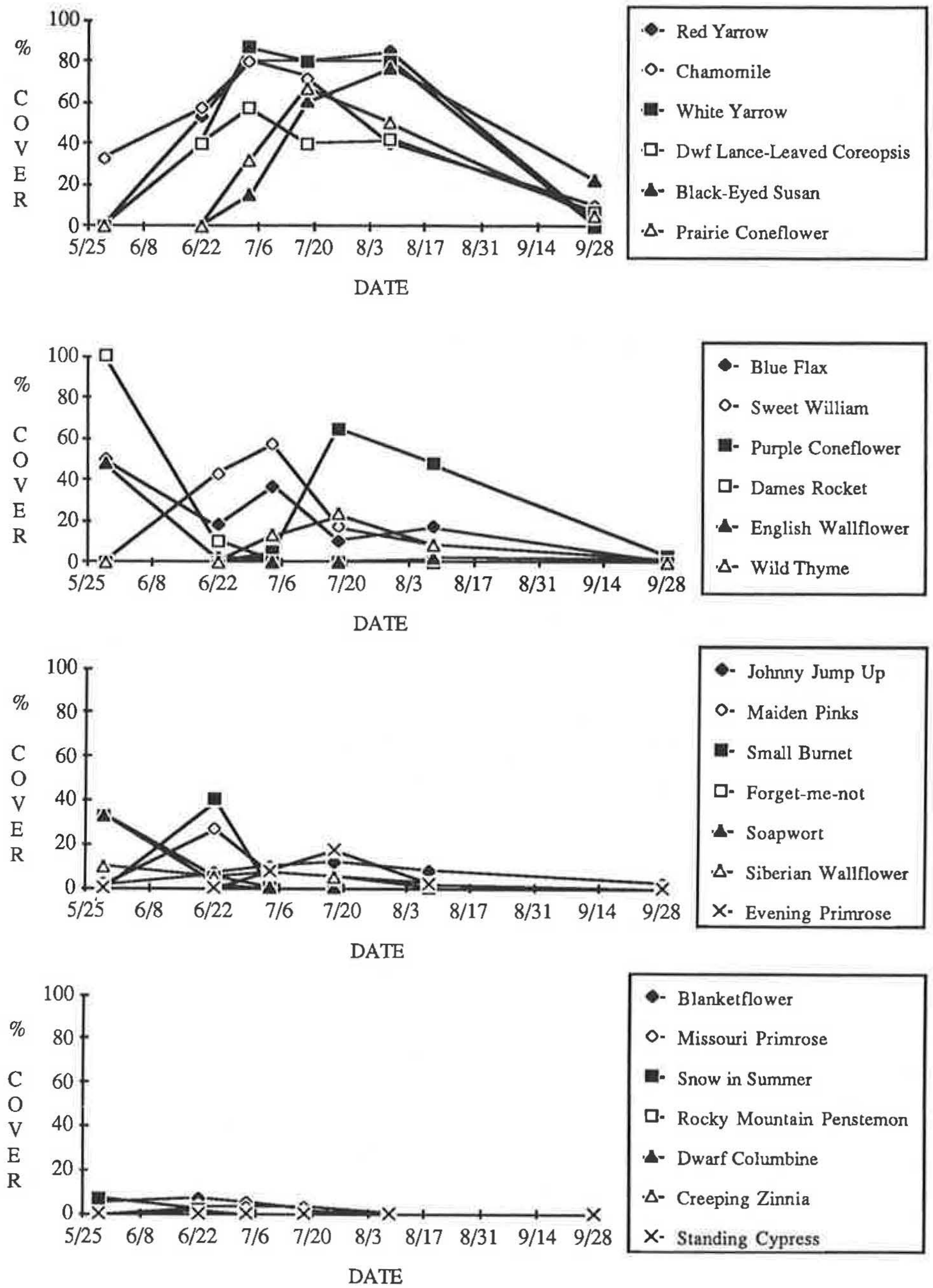



**FIGURE 8 Percent blossom cover for perennial wildflower species in 1989.**

1989 also provided impressive flowers during the 1989 ratings (Figure 8). Blue flax flowered the earliest and was already past its peak before the first rating date on May 29. Dames rocket had started flowering approximately a week before May 29 and was at its peak at that date. Chamomile, sweet William, lance-leaf coreopsis, and red and white yarrow were producing blooms on June 22 that continued into August. Black-eyed Susan, prairie coneflower, and purple coneflower were at peak flower production from July to early August.

From this information, wildflower mixes can be developed that combine annuals and perennials on the basis of their competitiveness, flowering time and duration, and growth characteristics. The performance of the individual species may be different when combined into a mix. Mixes will be tested under roadside conditions in several locations across Pennsylvania in 1990 to determine their performance over a variety of site and environmental conditions.

*Publication of this paper sponsored by Committee on Roadside Maintenance.*

# Computerized Data Base Management and Analysis System for Field-Collected Scour Data

RAVI NARASIMHAN, JAMES F. CRUISE, ANDRE P. STRAUTMANN, AND DEMETRI P. ARGIALAS

The scouring of the piers supporting highway bridges has come to be recognized as a major problem facing the highway engineer. It is becoming increasingly obvious that bridges that face possible scour situations must be monitored during their lifetimes. Much data will be collected during the monitoring phase. A computerized system for the storage, organization, analysis, and display of field-collected scour data is described. The system accepts input from the user, and on the basis of user specification, allows the data to be organized and viewed in a variety of formats. These formats include cross sections from scour measurements at selected locations upstream or downstream of the bridge, longitudinal profiles through a selected pier, and the temporal history of maximum scour activity within a specified area near the bridge. The data may be viewed in tabular as well as graphical formats. In addition, the available scour data were analyzed and used to develop regression equations that related long-term trends in channel degradation to flow and geometric variables. These equations were incorporated into the computer system so that the user could determine if observed trends in channel degradation were likely to continue.

The safety of highway bridges can be seriously impaired because of the undermining of the supporting piers by scour. The threat that potential bridge scour poses to the integrity of the highway system is becoming increasingly obvious. Recent failures in New York and Tennessee demonstrate the risk faced by many bridge structures. Potential scour must be taken into account during the design phase of bridge construction. However, it is also important that bridges that face possible scour situations be monitored during their lifetimes. Much data will be collected during the monitoring phase. These data must be organized and analyzed in the most efficient manner possible. The organization and analysis of field-collected scour data are the topics with which this paper is concerned.

Presently, the Louisiana Department of Transportation and Development (LADOTD) is monitoring about 80 bridges to detect possible danger of failure caused by pier scour. The scour surveys are done after flood events, or at specified time intervals during low water by fathometer and are reported on scour survey sheets that contain channel bottom elevations (NGVD) at selected locations upstream and downstream of the bridge piers. A schematic of a typical survey layout is shown in Figure 1. There are not sufficient data for either a full longitudinal profile or a full channel cross section to be plotted. In addition, no sediment data or discharge data are currently taken by LADOTD. Presently, the hydrographic survey data are plotted manually and then inspected visually and compared to past surveys to subjectively determine the relative severity of the scour situation at the particular location. This analysis can sometimes be very time consuming.

The monitoring program has been ongoing for about the past 12 years. A great deal of data (scour surveys) have been collected in that time period. A computerized system was created to store, organize, display, and analyze this historical survey data. This system, called the Louisiana Scour Analysis Management System (LASAMS), is capable of sorting and displaying the data base in a variety of formats as well as analyzing the data for the presence of trends in the observed data and relating these trends to bridge geometry and streamflow variables where possible.

## LASAMS

The scour analysis management system consists of three subsystems:

- A data entry and editor facility,
- A query facility that allows the data base to be sorted and viewed in a variety of tabular and graphical formats, and
- A scour analysis facility (performed through a query).

The system has been implemented on the Intergraph Interpro 32 workstation. The selection of Intergraph was based on the mutual availability of that system at both Louisiana State University (LSU) and LADOTD and the availability of software for data base and graphics manipulation. The Intergraph system also provided facilities for their integration. The choice of Intergraph for this project does not necessarily imply an endorsement of this product by the authors or LADOTD but merely that it was already available and possessed some of the necessary general capabilities. However, the system-provided software was not sufficient to perform the specific tasks required for this project. It does not possess the capability for a user-friendly interactive analysis of the data and the type of graphics necessary to meet the needs of this project. Therefore, a great deal of original programming (10,000 lines) in Fortran and Host Operations Language (HOL) was necessary. Thus, the final system is a composite of packages supplied by Intergraph and programs written for this project.

Department of Civil Engineering, Louisiana State University, Baton Rouge, La. 70803.

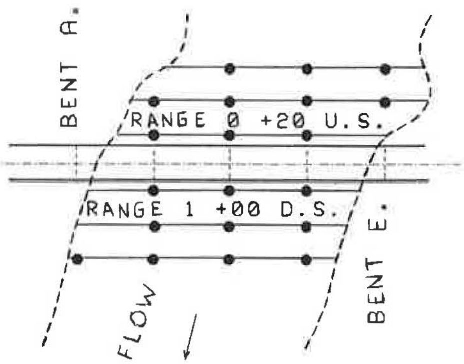



**FIGURE 1 Typical scour survey layout.**

LASAMS operates interactively in all three subsystems. The scour data base is constructed by entering data through the system-supplied editor, DMRS Worksheet Editor (DWE). This facility provides a spreadsheet-like interface to the data base. The specific attributes of the LASAMS data base and query facilities have been described by Narasimhan et al. (*1*) and in greater detail by Cruise and Argialas (*2*). The data base consists of eight entities that describe the bridge structure, channel conditions, and hydrographic survey. These entities and their descriptions are as follows:

- BRIDGE. This entity contains static information about bridges. The attributes of this entity include Name, ID Number, Route, etc.
- SOUNDING. This entity contains the actual sounding data. The attributes of this entity are
  - —Bridge number,
  - —Pier/bent number,
  - —Distance (from bridge),
  - —Position (upstream or downstream),
  - —Bottom elevation,
  - —Date of survey, and
  - —Comment.
- PIER. This entity contains information about the location of the piers of a bridge, their type, and their bottom elevations. This information has to be entered before the CROSS SECTION or HISTORY query can be made. The reference point for measuring the distances can be arbitrarily fixed as the left-most pier of the bridge (looking upstream).
- CROSS SECTION. This entity contains the cross section of a river near a bridge. These data must be obtained external to the scour surveys and are used for calculating the cross-sectional area, velocity, etc., required for scour analysis.
- BRIDGE NOTES. This entity contains comments about a bridge made by a survey team.
- PARISH CODE. This entity contains a list of parishes and their two-digit codes. This information is used only at the time of data entry for reference.
- PIER CODE. This entity contains information about the type of the pier and its corresponding code.
- STRUCTURE CODE. This entity contains the type of structure and its corresponding code.

Of course, the scour data base is continuously updated as new data become available.

## LASAMS ARCHITECTURE—QUERY SUBSYSTEM

The objective of LASAMS is to provide access to the archived scour data base and display scour data in tabular as well as graphical formats. In the Intergraph system, graphic data are stored and manipulated in a graphic design file. LASAMS therefore needs to access two resources: the scour data base and the scour graphics file. The scour data base can be accessed and manipulated programmatically. Although it is possible to create graphic elements programmatically, the display of graphics is possible only with the help of an interactive Intergraph program called Intergraph Command Environment (ICE). This necessitated the existence of two processes, ICE and Query, running concurrently.

The main function of ICE in LASAMS is to interact with the user and display the scour graphics file. The function of the Query program is to access the scour data base, and give the results of the queries made by the user. The Query program is also responsible for making graphical output of the query and plotting them on the scour graphics file. A scour trend analysis request is passed on to the scour analysis subsystem for processing. The system architecture is shown schematically in Figure 2. In reading the following description of Query architecture, it is helpful to refer to this figure.

The Intergraph system provides the facility to design sophisticated graphical interaction with the user by means of graphic menus, which are driven by graphic menu drivers. In the LASAMS system, the user specifies the criteria for query for filling up a form (the graphic menu). The criteria are checked for errors and compiled. The Query process is then spawned that waits for requests to serve. Once an error-free query is obtained, the menu driver transmits it to the Query process through a mailbox. The Query process now accesses the data base and gets the result of the query. The result is transmitted back to the menu driver through the mail box. The menu driver formats the result in the form of a table and displays

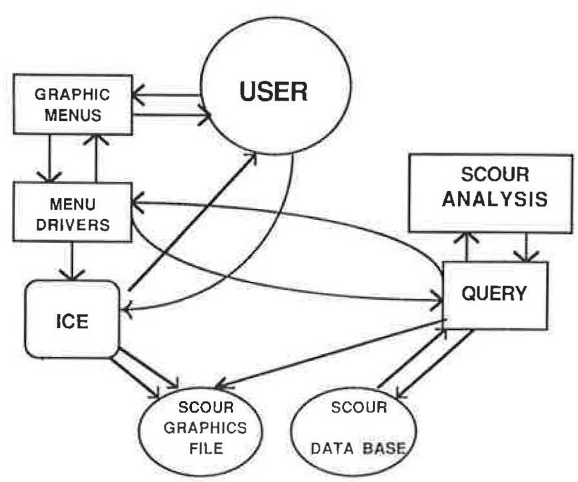



**FIGURE 2 LASAMS architecture.**

it to the user. If the user requests a graphical presentation of the results, this request is transmitted to the Query process. The Query process temporarily wrests control of the scour graphics file from the ICE process, creates graphic elements from the results of the query, and plots them on the graphics file. After plotting the graphics, control of the scour graphics file is returned to the ICE process, along with a completion message to the menu driver. The menu driver then displays the graphic results to the user with the help of ICE. Requests for scour analysis are also received by the Query process, but are passed on to the scour analysis system. The results of the analysis are received by the Query process and then transmitted to the menu driver. The architecture of LASAMS is such that it also facilitates direct interaction between the user and the powerful ICE program. This interaction allows a seasoned user of the Intergraph system to manipulate the scour graphics file using the full functionality of the ICE program, while still inside the LASAMS system.

The following sections describe each graphics menu available to the LASAMS user. These menus are grouped under the Query subsystem of the program.

### Main Menu QUERY

The user normally activates the main menu when logging on to the system. The available options are the following:

- REVIEW BRIDGES,
- SURVEY SHEET & PLAN,
- CROSS SECTION,
- LONGITUDINAL SECTION,

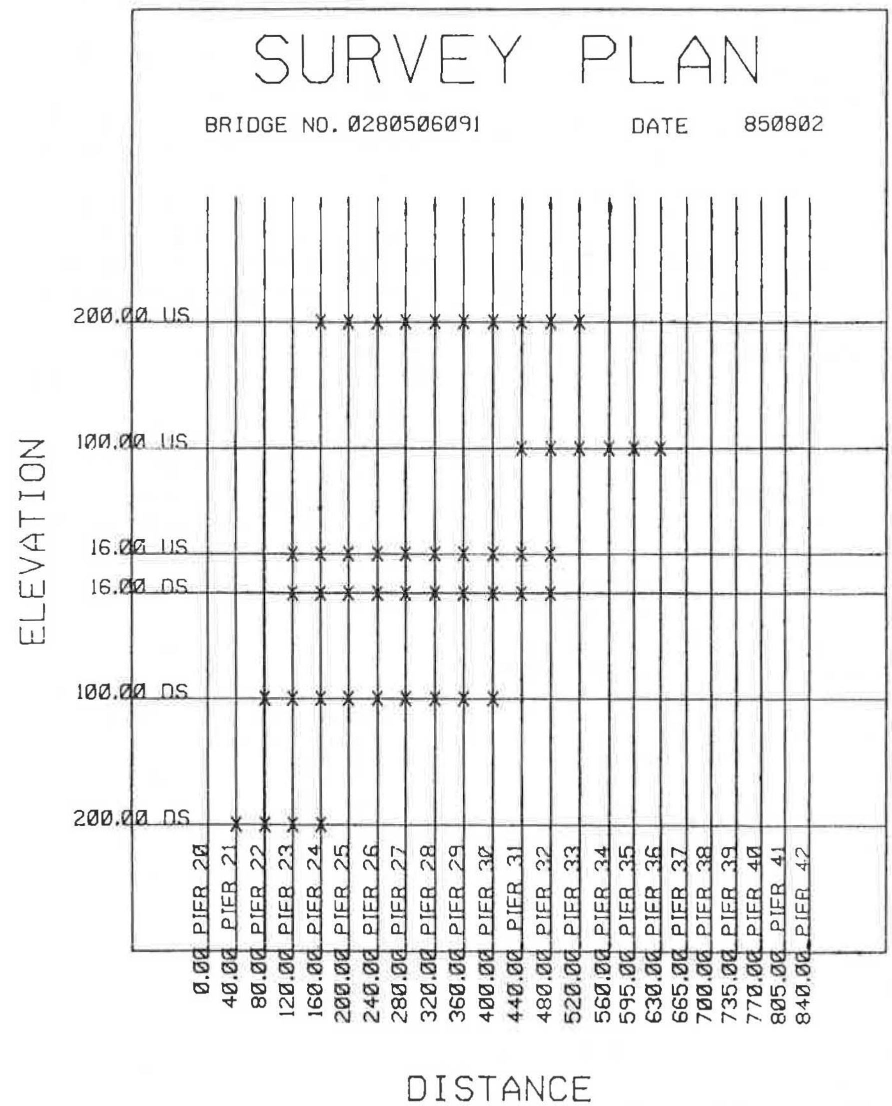



**FIGURE 3 Typical survey plan.**

- HISTORY & SCOUR PREDICTION, and
- END.

**Menu REVIEW BRIDGES**

This menu provides the user with information about bridges, such as the name of the bridge, number, route, structure, and frequency of survey. The specifications for this query are as follows:

- Bridge number,
- Parish,
- Route,
- District,
- Structure type, and
- Survey frequency (months).

The user can omit any or all of the specifications by just typing RETURN at the corresponding key entry fields.

**Menu SURVEY SHEET & PLAN**

Using this menu, the user can obtain the scour survey sheet and a plan of the survey for a bridge on a given date. This is the actual data reported by the survey crew. The specifications for this query are as follows:

- BRIDGE NUMBER and
- DATE OF SURVEY (in YYMMDD format).

If a graphical presentation of the survey plan is desired, the user selects the SURVEY PLAN option on the menu. A sample survey plan is shown in Figure 3.

**Menu CROSS SECTION**

Using this menu, the user can obtain the cross section of the scour data on a given date at a specified distance from the bridge. The specifications for this query are as follows:

- Bridge number;
- Distance from bridge (ft);

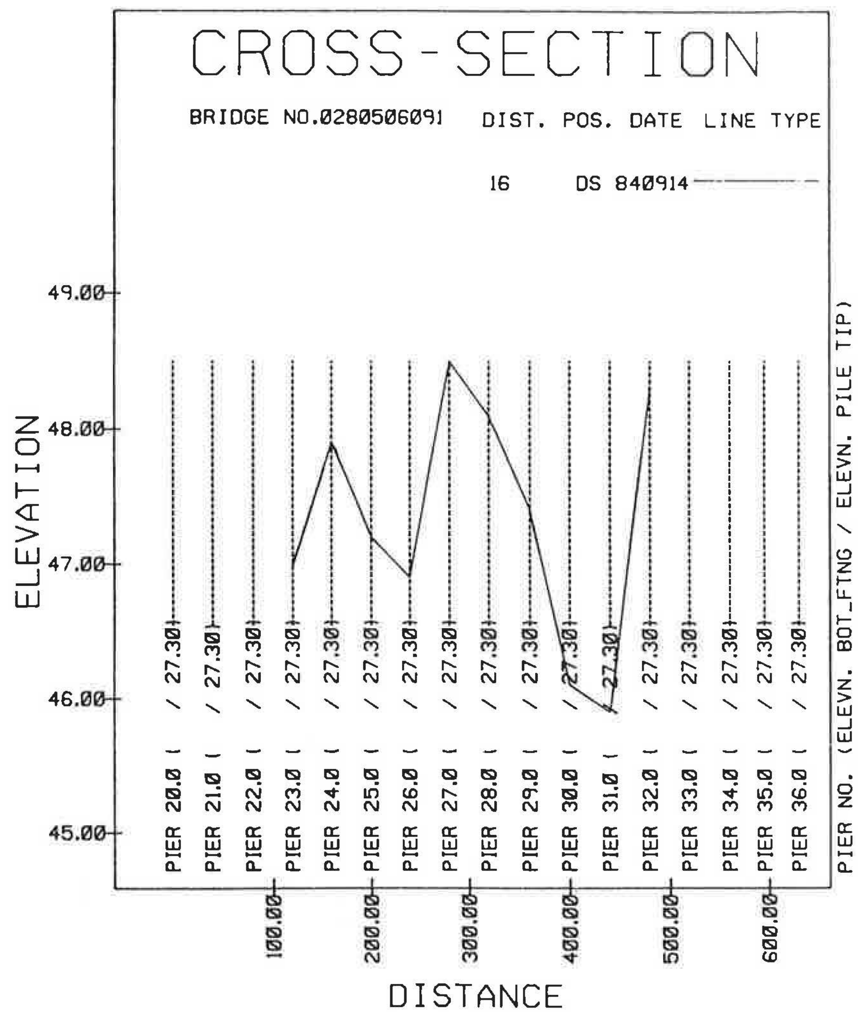



**FIGURE 4 Typical scour cross section.**

- Position (upstream US, centerline CL, or downstream DS); and
- Date of survey (in YYMMDD format).

A graphical presentation of the cross section is shown in Figure 4. Note that the cross sections corresponding to different survey dates can be overlayed on the screen for direct comparison.

## Menu LONGITUDINAL SECTION

Using this menu, the user can obtain the longitudinal section of the scour data on a given date along a given pier. The specifications for this query are as follows:

- Bridge number,
- Pier/bent number, and
- Date of survey (in YYMMDD format).

The longitudinal section is shown graphically in Figure 5. As in the case of cross sections, overlays of longitudinal sections can also be done for comparisons.

## Menu HISTORY AND SCOUR ANALYSIS

Using this menu, the user can obtain the variation of the maximum scour depth in a given area over a period of time and also get an estimate of the degradation trend likely to occur for a given flow history for some bridges. The specifications for the history query are as follows:

- Bridge number
- Upstream bound for area of interest
  —Distance from bridge (ft)
  —Position (upstream US, centerline CL, or downstream DS)

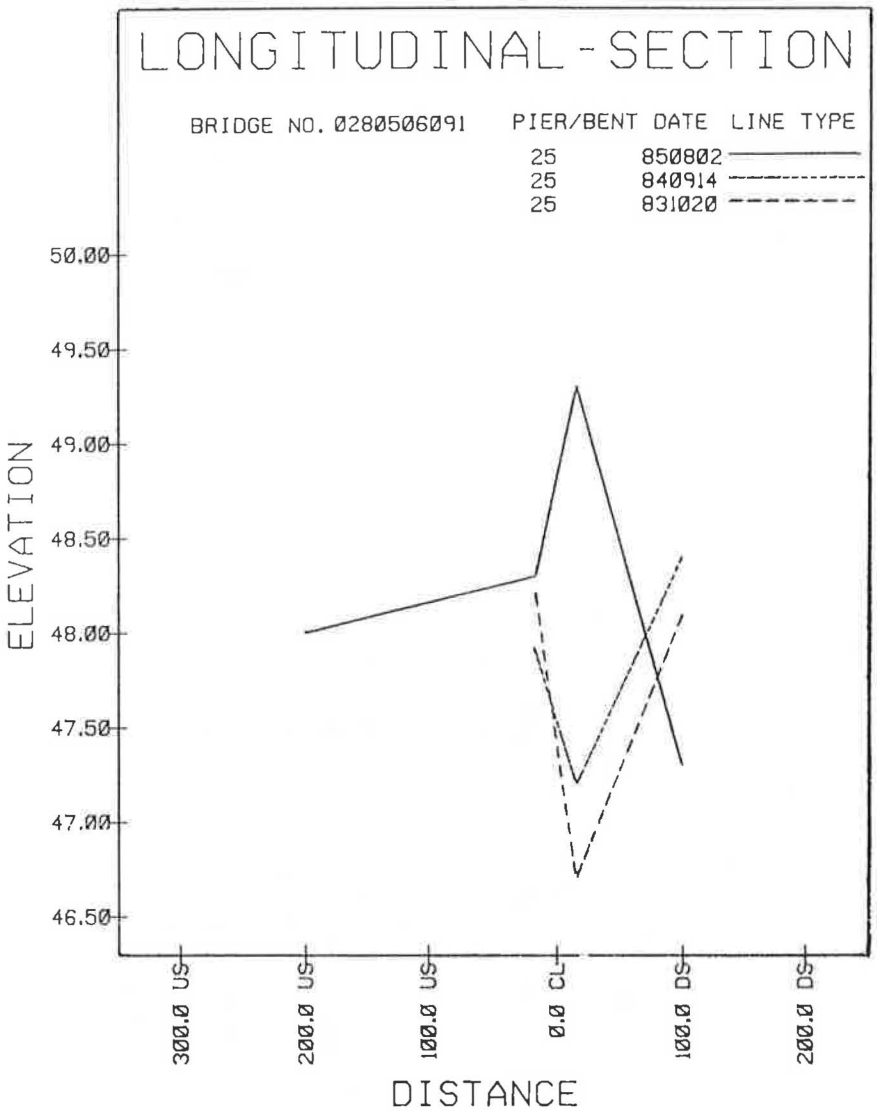



**FIGURE 5 Typical scour longitudinal section.**

- Downstream bound for area of interest
  —Distance from bridge (ft)
  —Position (upstream US, centerline CL, or downstream DS)
- Left bound, pier/bent number
- Right bound, pier/bent number
- Dates of interest (in YYMMDD format)
  —Initial date
  —Last date
- Bridge number
- Flood flow (in $ft^3$/sec)
- Stage (ft)

Under this option, the system scans the survey data within the specified bounds and chooses the maximum value for each survey. These values are then plotted versus the survey date as shown in Figure 6. A graphical display of scour trend (Figure 6) can be obtained by overlaying the results of trend prediction on a plot of a history query.

**End**

Selection of this option causes the program to end. The user has to return to the QUERY menu in order to end the program.

**TREND ANALYSIS**

The observed scour data are presently interpreted by visually comparing the current survey with past surveys at the same location. It is difficult to predict the relative severity of any trends that might be visually identified unless information about the flow history that caused the observed channel degradation is known. Scour analysis might be improved if this information could be incorporated into the system. For this reason, regression analyses were performed to relate trends in observed scour profiles with flow variables.

It is important for the reader to keep in mind the function of the regression equations. These equations in no way repre-

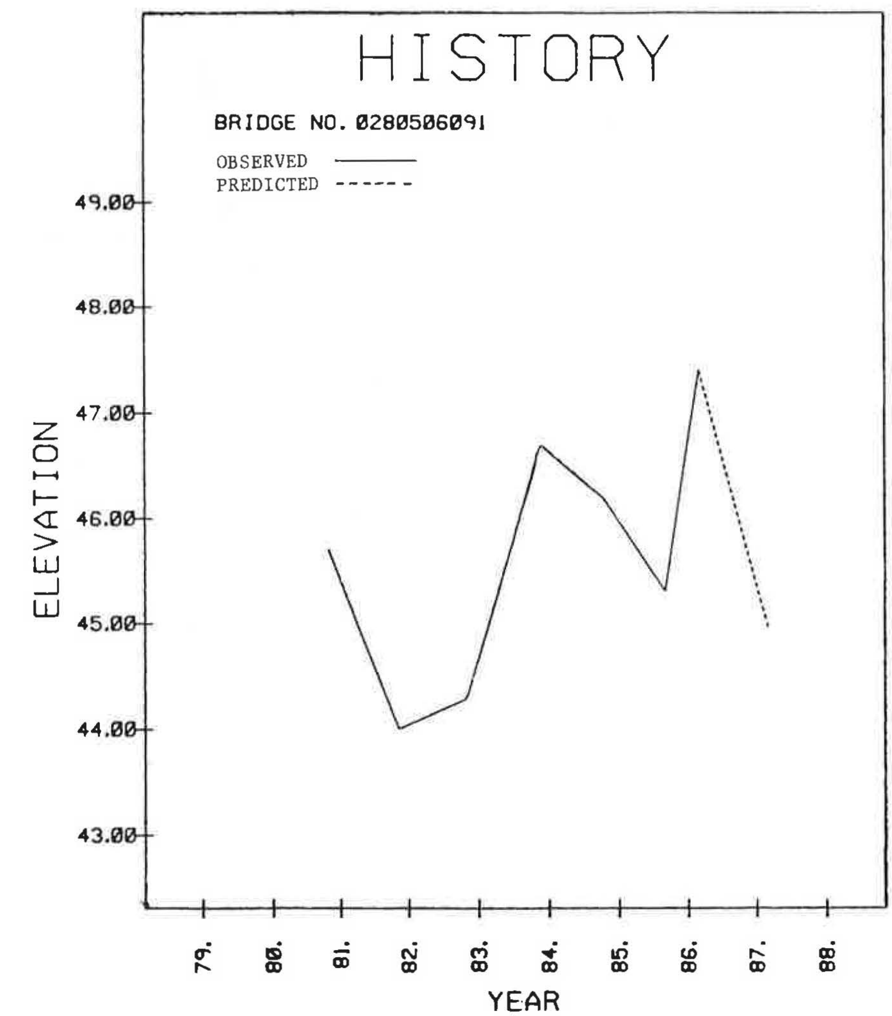



**FIGURE 6 Typical scour history and trend prediction plot.**

sent scour prediction equations. As stated previously, the scour surveys are normally done under low-water conditions. The stream velocities are usually negligible under these conditions and past studies have shown that there is little sediment movement in most Louisiana streams during low water. Thus, it is reasonable to assume that most bed degradation takes place during flood events. However, by the time the surveys are taken, much of the scour material has already been filled in during the receding flood waters. Therefore, the observed bed degradation is the residual or left over scour from the flood. There are no available data on maximum scour development during flood events for any of these streams. Therefore, it was not possible to develop equations to predict maximum scour or to check the validity of existing equations.

However, it is reasonable to assume that a relationship exists between the residual scour values and the flow variables that caused the original scour. The purpose in identifying this relationship is not to attempt to predict future maximum possible scour at any specific location, but to predict the future trend in channel degradation such that long term planning for bridge maintenance would be possible. For this reason, the equation must be site specific and incorporate the flow history of the specific stream under analysis. The U.S. Geological Survey maintains continuous stream gaging stations on 12 of the bridges in the current monitoring program. However, not all of these sites were suitable for analysis because of missing data or lack of channel control. Therefore, initial regression analyses were performed on four bridges. In all of these cases, it was found that a simple energy-based equation resulted in the best description of the observed channel degradation. This equation is

$$d_s = K_1 Y + K_2(V^2/2g)$$

where

$d_s$ = observed residual channel degradation (ft),
$V$ = average cross-sectional flow velocity (ft/sec),
$Y$ = average depth of flow in approach section (ft),
$g$ = acceleration of gravity (ft/sec$^2$), and
$K_1,K_2$ = site-specific regression coefficients.

In developing this equation, the flow variables for the most recent flood event preceding the scour survey were used. This formulation resulted in a better fit to observed data than did equations involving more variables including average discharge between surveys or maximum discharge between surveys. The average coefficient of determination ($R^2$) for the four bridges used in this analysis was 0.89. This equation has been incorporated into the scour history menu of the LASAMS program. It remains for LADOTD personnel to calibrate it for other bridges for which it will be applied. Calibration can be done using a regional flood study currently being completed for LADOTD (*3*) and the FHWA bridge backwater model.

## CONCLUSIONS

A computerized system has been developed for the storage, display, and analysis of field scour data. The program is currently in the implementation stage at the LADOTD. Once fully implemented, the program will greatly enhance the ability of department engineers to efficiently analyze these data. The historic survey data for more than 80 bridges will be transferred from paper files to the computer system. This data base can easily be updated as new surveys are taken. The data can be sorted and visually compared on the screen in a variety of graphical formats. For bridges at which trend analyses are desired, a trend analysis formula has been developed to relate trends in the data to flow variables. This formula should greatly aid in the objective analysis of the scour trends. The LASAMS package should facilitate and enhance the scour monitoring program at LADOTD and help ensure the safety of Louisiana highway bridges from scour failures.

## ACKNOWLEDGMENTS

This project was funded by the Louisiana Transportation Research Center (LTRC) under the project, "Automation of Scour Analysis at Louisiana Bridge Sites." The guidance and support of Babak Naghavi and Steve Cumbaa of LTRC and Jack Manno of the Hydraulics Section of LADOTD are also gratefully acknowledged. LASAMS was constructed using the facilities of the LSU Remote Sensing and Image Processing Laboratory, whose director, Charles Harlow, was always helpful and supportive.

## REFERENCES

1. R. Narasimhan, J. F. Cruise, D. P. Argialas, A. P. Strautmann, K. Arora, and D. Barbe. Louisiana Scour Analysis Management System (LASAMS). *Proc., American Water Resources Association National Conference, 1989*, Bethesda, Md., pp. 11A-9–11A-19.
2. J. F. Cruise and D. P. Argialas. *Automation of Scour Analysis at Louisiana Bridge Sites*. Final Report, Louisiana Transportation Research Center, Baton Rouge, La., 1989.
3. J. F. Cruise, B. Naghavi, and S. Ekanayake. *Prediction of Flood Quantiles at Ungaged Watersheds in Louisiana*. Report FHWA-LA-229. Louisiana Transportation Research Center, Baton Rouge, La., 1990, 68 pp.

*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*

# Comparative Evaluation of Three Estimators of Log Pearson Type 3 Distribution

BABAK NAGHAVI, JAMES F. CRUISE, AND KISHORE ARORA

Three moment-based estimation procedures for log Pearson Type 3 (LP3) parameters were compared using observed stream data samples from Louisiana and its neighboring states. The methods of direct moments, log-transformed moments, and mixed moments were compared in descriptive capabilities on the basis of computed root mean square deviation (RMSD) and mean absolute deviation (MAD) of the standardized variate. Using these performance indices, the most robust estimation method was sought. In many cases, depending on sample skewness, significant differences existed between the descriptive capabilities of these methods. However, no method performed in a clearly superior manner across the entire range of data. These results can be used in conjunction with previous Monte Carlo studies focusing on the predictive ability of these procedures to determine the most reliable moment-based estimation procedures.

The log Pearson Type 3 (LP3) distribution is one of the most widely used distributions in hydrology, particularly in flood frequency analysis, as recommended by many governmental agencies in the United States. Many highway drainage structures are designed under the assumption that flood discharges follow this distribution. The LP3 distribution was first recommended by the U.S. Water Resources Council (WRC) in 1967 as the base method of flood frequency analysis in the United States. Since then, a great deal of interest has been generated in this distribution. The LP3 distribution has been extensively discussed by Bobee (*1*), Bobee and Robitaille (*2*), Condie (*3*), Rao (*4*), and many others.

Much attention has been focused on parameter estimation. Bobee (*1*) suggested an estimation method that was based on the first three moments of raw data, the method of direct moments (MDM), whereas the WRC (*5*) recommended an estimation method that was based on the corresponding moments of the log-transformed data. Condie (*3*) proposed an estimation method that was based on maximum likelihood estimation (MLE) theory. Rao (*4*) proposed the method of mixed moments (MIX), which uses sample estimates of the means of the raw and log-transformed data and the standard deviation of the raw data in estimating the parameters. Singh and Singh (*6*) used the principle of maximum entropy (POME) to estimate the parameters for the LP3 distribution. Arora and Singh (*7*) and Ashkar and Bobee (*8*), among others, compared performance of various methods of parameter estimation via Monte Carlo simulation. In terms of root mean square error (RMSE) and bias, Arora and Singh (*7*) found that the MIX and MDM methods were clearly superior to the WRC method for simulated LP3 samples. Ashkar and Bobee (*8*) compared four versions of the method of moments and observed that the MDM method performed better than the other three. More recently, Bobee and Ashkar (*9*) studied several variations of the method of moments and concluded that different versions of the methodology could result in significantly different fits to the data series. They also concluded that no one version of moment-based methods can be considered best for all applications.

MLE of the LP3 parameters has been found to be computationally difficult and results in multiple roots of the location parameter (*10*). Arora and Singh (*10*) also found that MLE performed poorly in terms of RMSE and bias in comparison with moment-based methods on the basis of small Monte Carlo–generated samples. A large amount of CPU time was required by the search routines for MLE estimation of LP3 parameters. Thus, MLE techniques do not appear to be well suited to the estimation of parameters of the LP3 distribution. In the case of the LP3 distribution, the maximum entropy procedure and the MIX method lead to the same parameter estimation equations (*11*). Therefore, moment-based methods may be the most practical and computationally efficient procedures for estimating LP3 parameters and quantiles.

The MDM estimates the parameters of the LP3 distribution directly from the untransformed data. In this method, the observed data are equally weighted in the estimation of the parameters. Thus, this procedure maintains the significance of the larger sample values because the spatial relationship among the real data is preserved.

Conversely, the WRC method weights the logarithms of the observed data equally in parameter estimation. Therefore, in this method, the larger sample values are given less significance because of the transformation into log space before the sample statistics were computed. The WRC method has been criticized because of the sampling properties of the coefficient of skewness. This statistic has been shown to be significantly downward biased (*12*) and algebraically bounded (*13*) and possesses a large sampling variability (*14*). Studies by Wallis and Wood (*15*), Arora and Singh (*7*), and Ashkar and Bobee (*8*) have reported the poor performance of the WRC method on the basis of Monte Carlo analyses. The method of direct moments also requires the estimation of the third moment from the data sample.

B. Naghavi, Louisiana Transportation Research Center, 4101 Gourrier Avenue, Baton Rouge, La. 70808. J. F. Cruise and K. Arora, Department of Civil Engineering, Louisiana State University, Baton Rouge, La. 70803.

The MIX method combines the moment equations in real and log-transformed space in parameter estimation. MIX avoids the use of sample statistics (such as skew) based on third-moment estimates that are susceptible to large sampling errors.

Previous comparisons were based on Monte Carlo simulations in which the data were generated from known distributions, typically the LP3. However, in real-world situations, the population distributions are unknown. Therefore, a comparison of the most popular parameter estimation procedures using real data seems timely. Although it is recognized that a good fit to observed data is not a sufficient reason for accepting a particular method, an adequate fit to the observed data is a necessary condition for the acceptance of a procedure. Cunnane (*16*) compares the relative importance of predictive and descriptive abilities of flood prediction techniques. He concludes that neither attribute is more important than the other; indeed, the two characteristics are complementary. In order for a particular technique to be useful, it must possess both predictive and descriptive abilities. Predictive capabilities are usually determined from Monte Carlo studies of a particular method or distribution, whereas descriptive capabilities can be determined from analyses based on real-world data, with the added advantage of unknown population distributions. Thus, studies such as the one reported here can be used in conjunction with the Monte Carlo studies previously reported to aid in the selection of the most reliable estimation technique for LP3 parameters and quantiles. If events of small recurrence intervals ($T \leq 25$ years) are to be estimated, the method with the superior descriptive capability may be preferred, because events of this magnitude will usually already be recorded in the systematic record. In highway drainage work, many times structures are designed for small recurrence intervals whose quantiles may already have been recorded. In these cases, the interpretive ability of the method may be of paramount importance. However, if events of larger recurrence intervals are to be estimated, then some descriptive ability may be sacrificed to obtain improved predictive ability. In this study, three moment-based methods (WRC, MIX, and MDM) are compared, using gauge stations in Louisiana (87 stations) and its neighboring states (6 stations) with unknown flood distributions.

## PROPERTIES OF LOG PEARSON TYPE 3 (LP3) DISTRIBUTION

The probability density of the LP3 is

$$f(x) = \frac{1}{|a|x\Gamma(b)}\left[\frac{\ln(x-c)}{a}\right]^{b-1} \exp\left[-\frac{\ln(x-c)}{a}\right] \tag{1}$$

where

$x$ = raw (untransformed) flood data,
$a$ = LP3 scale parameter,
$b$ = LP3 shape parameter, and
$c$ = LP3 location parameter.

The parameter $b$ is always positive and $\Gamma$ is the gamma function. LP3 density function is flexible and can take many different forms. The mean, variance, and skewness coefficient of the variate $y = \ln(x)$ are given by

Mean

$$\mu = c + ab \tag{2}$$

Variance

$$\sigma^2 = ba^2 \tag{3}$$

Skew

$$\gamma = \frac{|a|}{a}\frac{2}{b^{1/2}} \tag{4}$$

The moments of $x$ about the origin are given by (*1*)

$$\mu_r' = \frac{\exp(rc)}{(1-ra)^b} \qquad 1 - ra > 0,\ r = 1, 2, \text{ and } 3 \tag{5}$$

If $a > 0$, then $\gamma_y > 0$; therefore $y$ must be positively skewed such that $f(y)$ is lower bounded ($c \leq y < +\infty$). In this case, $x$ must also be positively skewed, thus $x$ also possesses a lower bound [$\exp(c) \leq x < +\infty$] (*4*). When $a < 0$, then $\gamma_y < 0$ such that $y$ is negatively skewed and upper bounded, that is, $-\infty < y \leq c$. In this case, $x$ either can be positively or negatively skewed, depending on the values of the parameters $a$ and $b$, but $x$ is upper bounded [$0 < x < \exp(c)$]. For this case, the density function $f(x)$ may be defined as zero at $x = 0$ (*4*).

The overall geometric shape of the LP3 distribution is governed by the parameters $a$ and $b$ (*1,4*). The LP3 distribution degenerates to the log normal distribution when the parameters $a$ and $b$ approach zero and infinity, respectively.

### Fitting the LP3 Distribution by the Method of Logarithmic Moments (WRC)

This method estimates $a$, $b$, and $c$ by applying the method of moments to the log-transformed data. Equations 2–4 are used for estimating the parameters where $\mu$, $\sigma^2$, and $\gamma$ are substituted by the mean, variance, and skewness coefficient estimates of the log-transformed sample.

### Fitting the LP3 Distribution by the MIX Method

Rao (*4*) proposed the MIX method for LP3 with the objective of avoiding use of the sample skewness coefficient in parameter estimation. The MIX method preserves the sample mean and variance of raw data ($\bar{x}$, $S_x^2$) and sample mean of the log-transformed data ($\bar{y}$). The MIX parameter estimation equations are

$$\bar{y} = c + ab \tag{6}$$

$$\bar{x} = \frac{\exp(c)}{(1-a)^b} \tag{7}$$

$$S_x^2 = \exp(2c)\left[\frac{1}{(1-2a)^b} - \frac{1}{(1-a)^{2b}}\right] \tag{8}$$

A method of solution of Equations 6–8 has been devised by Arora and Singh (*7*).

Eliminating $c$ by combining Equations 6 and 7,

$$\bar{y} - \ln(\bar{x}) = b[a + \ln(1 - a)] \tag{9}$$

Again, $c$ can be eliminated by combining Equations 7 and 8.

$$\ln\left(\frac{S_x^2 + \bar{x}^2}{\bar{x}^2}\right) = 2b \ln(1 - a) - b \ln(1 - 2a) \tag{10}$$

Combining Equations 9 and 10,

$$\frac{2 \ln(1 - a) - \ln(1 - 2a)}{\ln(1 - a) + a} = P \tag{11}$$

where $P$ can be found from sample estimates of $\bar{x}$, $\bar{y}$, and $S_x^2$:

$$P = \frac{\ln[(S_x^2 + \bar{x}^2)/\bar{x}^2]}{(\bar{y} - \ln \bar{x})} \tag{12}$$

The left-hand side of Equation 11 is defined for $a < ½$. Therefore, values of $a$ can be found using a trial-and-error search method or by the Newton-Raphson iteration. Alternatively, $a$ can be determined from interpolation of the $a$–$P$ table given by Arora and Singh (*7*). Parameters $b$ and $c$ can then be estimated from Equations 9 and 6, respectively.

### Fitting the LP3 Distribution by the MDM

The MDM applies the method of moments directly to the raw data to determine parameters $a$, $b$, and $c$. Substituting the first three sample moment estimates in Equation 5 yields three simultaneous equations:

$$\ln \mu_1' = c - b \ln(1 - a) \tag{13}$$

$$\ln \mu_2' = 2c - b \ln(1 - 2a) \tag{14}$$

$$\ln \mu_3' = 3c - b \ln(1 - 3a) \tag{15}$$

These equations are solved in a manner given in Arora and Singh (*7*) that is similar to the method proposed by Bobee (*1*). Equations 13–15 can be rearranged to give

$$\frac{\ln \mu_3' - 3 \ln \mu_1'}{\ln \mu_2' - 2 \ln \mu_1'} = \frac{3 \ln(1 - a) - \ln(1 - 3a)}{2 \ln(1 - a) - \ln(1 - 2a)} \tag{16}$$

The right-hand side of Equation 16 is defined for $a < ⅓$. In practice, $B$ is obtained from the sample estimates of the first three moments about the origin:

$$B = \frac{\ln \mu_3' - 3 \ln \mu_1'}{\ln \mu_2' - 2 \ln \mu_1'}$$

With $B$ calculated, the value of $a$ follows from Equation 16 using a trial-and-error search method, Newton-Raphson iteration, or interpolation of the $a$–$B$ table given by Bobee (*1*) or Arora and Singh (*7*). Parameters $b$ and $c$ can then be estimated from Equations 13 and 14.

## COMPARATIVE EVALUATION OF THE THREE METHODS

A total of 114 gauge stations with 20 years or more of record were initially available for use in this comparative study. Analysis of these data revealed that the records of 10 gauge stations were contaminated by diversions, regulation, backwater, etc., and thus were eliminated from further analysis. The pertinent data for the 93 remaining stations are presented in Table 1. Sample skew of the untransformed data varied from −0.40 to 6.18, and sample coefficient of variation varied from 0.29 to 1.75. This range represents a fairly broad range in the statistical characteristics of the available data samples. Although all of the data are drawn from one region of the United States, because of the range in skewness of the data base the results may hold significance for other regions.

Grubbs and Beck outlier analysis (*17*) at 10 percent significance level ($\alpha = 0.10$) was conducted, and 10 stations with single outliers were identified. Data with and without the outlier were analyzed for these sites. The performance of the three methods was evaluated using performance indices similar to those used by Singh and Singh (*6*), among others. These are standardized root mean square deviation (SRMSD) given by

$$\text{SRMSD} = \left[\frac{1}{N}\sum_{i=1}^{N}\left(\frac{\hat{x}_i - x_i}{\bar{x}}\right)^2\right]^{1/2} \tag{17}$$

and standardized mean absolute deviation (SMAD) given by

$$\text{SMAD} = \frac{1}{N}\sum_{i=1}^{N}\left|\frac{\hat{x}_i - x_i}{\bar{x}}\right| \tag{18}$$

where $N$ is the sample size $(x_1, x_2, \ldots, x_N)$,

$$\bar{x} = \frac{1}{N}\sum_{i=1}^{N} x_i,$$

and $\hat{x}_i$ is an estimate of $x_i$ obtained from $F^{-1}[p(x_i)]$; $p(x_i)$ is approximated by the Weibull plotting position: $p(x_i) = m_i/(N + 1)$, where $m_i$ is the rank of $x_i$ in descending order. The Weibull is an unbiased empirical estimate of the quantile probability and is the most widely used plotting position formula in hydrology. In this analysis, $F^{-1}$ values were approximated numerically, and LP3 quantiles were obtained using a routine that translates the LP3 to a chi-squared random variate.

The performance indices used here are different from those used in the previous references in that the deviations between the predicted and observed variate are standardized by the sample mean rather than the observed value itself. In this way, every observed value of the variate is given equal weight in the computation of the performance index. Results of performance evaluation using SRMSD and SMAD are presented in Tables 2 and 3, respectively. The maximum percent differences were obtained from the differences between the

TABLE 1 PERTINENT DATA OF WATERSHEDS

| Gage Station | Area km² | No. of Obs. | Skew Coefficient | Coefficient of Variation |
|---|---|---|---|---|
| 02491500 | 2,564 | 66 | 2.12 | 0.75 |
| 02492000 | 3,139 | 50 | 3.13 | 0.85 |
| 07344450 | 207 | 31 | 2.38 | 1.05 |
| 07348700 | 1,566 | 30 | 2.23 | 0.93 |
| 07349500 | 1,413 | 49 | 1.53 | 0.68 |
| 07351500 | 171 | 49 | 2.28 | 0.91 |
| 07352000 | 399 | 47 | 1.60 | 0.85 |
| 07351000 | 205 | 43 | 1.17 | 0.72 |
| 07366200 | 538 | 32 | 2.86 | 1.07 |
| 07371500 | 919 | 49 | 1.61 | 0.77 |
| 07372200 | 4,914 | 30 | 2.33 | 0.88 |
| 07373000 | 132 | 46 | 1.92 | 1.11 |
| 07375000 | 267 | 44 | 1.92 | 0.96 |
| 07375500 | 1,672 | 49 | 2.15 | 0.82 |
| 07376000 | 639 | 47 | 1.39 | 0.75 |
| 07376500 | 207 | 44 | 1.20 | 0.59 |
| 07377000 | 1,501 | 39 | 0.80 | 0.73 |
| 07377500 | 375 | 45 | 0.70 | 0.70 |
| 07378000 | 735 | 44 | 1.22 | 0.57 |
| 07378500 | 3,313 | 49 | 1.57 | 0.71 |
| 07381800 | 176 | 33 | 1.26 | 0.78 |
| 07382000 | 621 | 50 | 6.18 | 1.75 |
| *07382000 | 621 | 49 | 2.89 | 0.70 |
| 08010000 | 339 | 49 | 0.77 | 0.49 |
| 08012000 | 1,364 | 49 | 2.53 | 0.72 |
| *08012000 | 1,364 | 48 | 2.47 | 0.63 |
| 08013000 | 1,291 | 44 | 1.62 | 0.75 |
| 08013500 | 1,949 | 49 | 2.21 | 0.72 |
| 08014500 | 1,320 | 48 | 4.83 | 1.30 |
| *08014500 | 1,320 | 47 | 1.30 | 0.74 |
| 08015500 | 4,399 | 49 | 3.43 | 0.84 |
| *08015500 | 4,399 | 48 | 1.34 | 0.59 |
| 08014000 | 443 | 27 | 2.10 | 0.90 |
| 08025500 | 383 | 31 | 2.03 | 1.16 |
| 08028000 | 945 | 36 | 1.91 | 1.15 |
| 02490105 | 188 | 22 | 1.30 | 0.82 |
| 07375222 | 119 | 22 | -0.18 | 0.56 |
| 07380160 | 52 | 33 | 0.40 | 0.48 |
| 07375170 | 228 | 20 | 1.18 | 0.63 |
| 07377300 | 2,290 | 35 | 1.27 | 0.61 |
| 07376600 | 36 | 32 | 0.07 | 0.37 |
| 07375480 | 236 | 20 | 1.61 | 0.85 |
| 02491700 | 114 | 20 | 1.56 | 0.95 |
| 02491350 | 109 | 21 | 1.39 | 0.85 |
| 07375800 | 232 | 32 | 2.62 | 1.11 |
| 07375307 | 135 | 22 | 1.61 | 1.06 |
| 07373500 | 91 | 21 | 0.73 | 0.61 |
| 07364300 | 702 | 24 | 2.18 | 0.94 |
| 07369500 | 800 | 52 | 0.05 | 0.29 |
| 07386500 | 49 | 28 | 0.80 | 0.43 |
| 08011800 | 114 | 24 | 1.21 | 0.67 |
| 08014200 | 244 | 37 | 3.39 | 1.06 |
| 07353500 | 122 | 26 | 1.73 | 1.11 |
| 07372500 | 238 | 31 | 4.16 | 1.20 |
| *07372500 | 238 | 30 | 1.46 | 0.64 |
| 07370750 | 123 | 30 | 2.28 | 0.86 |
| 07372110 | 62 | 23 | 2.24 | 1.22 |
| 07372000 | 1,694 | 42 | 0.92 | 0.67 |
| 07370500 | 702 | 30 | 1.32 | 0.80 |
| 07370000 | 2,025 | 60 | 0.49 | 0.40 |
| 07367250 | 23 | 20 | 1.88 | 1.10 |
| 07366403 | 1 | 22 | 1.95 | 1.03 |
| 07366420 | 293 | 22 | 3.16 | 1.20 |
| 07365000 | 919 | 28 | 1.68 | 0.80 |
| 07364870 | 122 | 22 | 1.47 | 0.74 |
| 07365500 | 461 | 30 | 4.02 | 1.26 |

TABLE 1 (*continued on next page*)

TABLE 1 (continued)

| Gage Station | Area $km^2$ | No. of Obs. | Skew Coefficient | Coefficient of Variation |
|---|---|---|---|---|
| *07365500 | 461 | 29 | 1.47 | 0.70 |
| 07366000 | 1,197 | 43 | 3.37 | 1.11 |
| 07364700 | 365 | 22 | 2.59 | 1.43 |
| 08016600 | 213 | 38 | 1.03 | 0.63 |
| 08028700 | 34 | 26 | 2.87 | 0.70 |
| *08028700 | 34 | 25 | 1.26 | 0.46 |
| 08014600 | 68 | 20 | 1.63 | 0.88 |
| 08013800 | 27 | 21 | 1.46 | 0.77 |
| 08013610 | 1 | 22 | -0.40 | 0.30 |
| 07354000 | 55 | 30 | 0.39 | 0.49 |
| 07353990 | 97 | 22 | 1.54 | 1.01 |
| 08016800 | 458 | 31 | 2.69 | 0.81 |
| 08016400 | 383 | 39 | 1.62 | 0.73 |
| 08015000 | 616 | 31 | 1.66 | 0.97 |
| 07352500 | 1,096 | 43 | 1.08 | 0.70 |
| 02490000 | 31 | 20 | 1.90 | 1.01 |
| 07348725 | 86 | 22 | 0.70 | 0.63 |
| 07348800 | 173 | 24 | 2.02 | 0.88 |
| 07347000 | 300 | 25 | 2.37 | 0.50 |
| *07347000 | 300 | 24 | 0.79 | 0.34 |
| 07362100 | 997 | 49 | 3.01 | 1.03 |
| 07364190 | 3,030 | 45 | -0.36 | 0.35 |
| 07365800 | 466 | 29 | 3.89 | 1.68 |
| *07365800 | 466 | 28 | 4.02 | 1.17 |
| 07373550 | 1 | 30 | 0.61 | 0.41 |
| 08014800 | 311 | 24 | 1.50 | 0.74 |
| 08025850 | 25 | 20 | 2.30 | 0.98 |
| 08024060 | 8 | 24 | 0.09 | 0.43 |
| 08023000 | 250 | 28 | 1.39 | 0.74 |
| 07351700 | 50 | 26 | 4.60 | 1.53 |
| *07351700 | 50 | 25 | 0.02 | 0.54 |
| 07368500 | 109 | 28 | 0.42 | 0.34 |
| 07364500 | 4,260 | 52 | 0.06 | 0.36 |
| 02492360 | 453 | 21 | 0.95 | 0.61 |
| 08031000 | 216 | 34 | 1.31 | 0.68 |
| 08030000 | 179 | 32 | 1.55 | 0.64 |
| 08029500 | 332 | 36 | 2.55 | 1.15 |

*Run without the outliers.

methods with the largest and smallest SRMSD (Table 2) and SMAD (Table 3).

## DISCUSSION OF RESULTS

The most robust estimation technique in terms of descriptive ability was determined from among the moment-based procedures. The robust procedure is that which performs best across all variation in sample statistics. Kuczera (*18*) discusses two possible measures of robustness: minimax RMSD and minimum average RMSD. Based on the minimax criterion, the preferred estimator is the one whose maximum RMSD for all cases is minimum. The minimum average criterion is to select the estimator whose RMSD average over the test cases is minimum.

Table 4 presents the results of robustness studies using the two performance indices SRMSD and SMAD from Equations 17 and 18. The table shows the minimum, average, and maximum values of each performance index both for with and without (parenthesis) outlier cases. The results indicate that if the SRMSD index is preferred, the MDM is superior both under the average SRMSD and minimax SRMSD criteria. If the SMAD index is used, however, then the WRC method is the most robust estimator under both criteria. The table also shows that removal of the outliers has a large effect on the maximum values of the two indices, some effect on the averages, and of course, no effect on the minimum values.

From Table 2, in most cases when outliers were removed, the WRC method performed better than MDM and MIX by the SRMSD index. This result implies that the WRC method may be more sensitive to the presence of outliers than the other methods within the range of skewness characteristic of the data base.

Further analyses were performed by examining the performance of the different methods within particular ranges of

TABLE 2 SRMSD TEST RESULTS FOR THE THREE LP3 FITTING METHODS

| Gage Station | SRMSD WRC | SRMSD MIX | SRMSD MDM | Method(s) with Min. SRMSD | Max. Diff. % |
|---|---|---|---|---|---|
| 02491500 | 0.208 | 0.210 | 0.211 | WRC | 1.4 |
| 02492000 | 0.307 | 0.296 | 0.296 | MIX/MDM | 3.7 |
| 07344450 | 0.328 | 0.354 | 0.326 | MDM | 8.6 |
| 07348700 | 0.250 | 0.281 | 0.272 | WRC | 12.4 |
| 07349500 | 0.135 | 0.141 | 0.144 | WRC | 6.7 |
| 07351500 | 0.200 | 0.137 | 0.137 | MIX/MDM | 46.0 |
| 07352000 | 0.100 | 0.138 | 0.135 | WRC | 38.0 |
| 07351000 | 0.142 | 0.124 | 0.126 | MIX | 14.5 |
| 07366200 | 0.446 | 0.437 | 0.423 | MDM | 5.4 |
| 07371500 | 0.233 | 0.198 | 0.200 | MIX | 17.7 |
| 07372200 | 0.253 | 0.260 | 0.261 | WRC | 3.2 |
| 07373000 | 0.178 | 0.208 | 0.187 | WRC | 16.9 |
| 07375000 | 0.155 | 0.202 | 0.198 | WRC | 30.3 |
| 07375500 | 0.182 | 0.199 | 0.201 | WRC | 10.4 |
| 07376000 | 0.107 | 0.126 | 0.126 | WRC | 17.8 |
| 07376500 | 0.080 | 0.093 | 0.095 | WRC | 18.8 |
| 07377000 | 0.148 | 0.113 | 0.104 | MDM | 42.3 |
| 07377500 | 0.178 | 0.115 | 0.089 | MDM | 100.0 |
| 07378000 | 0.104 | 0.104 | 0.107 | WRC/MIX | 2.9 |
| 07378500 | 0.122 | 0.139 | 0.141 | WRC | 15.6 |
| 07381800 | 0.106 | 0.130 | 0.121 | WRC | 22.6 |
| 07382000 | 1.096 | 1.150 | 1.004 | MDM | 14.5 |
| *07382000 | 0.199 | 0.223 | 0.226 | WRC | 13.6 |
| 08010000 | 0.087 | 0.065 | 0.067 | MIX | 33.8 |
| 08012000 | 0.208 | 0.235 | 0.235 | WRC | 13.0 |
| *08012000 | 0.185 | 0.199 | 0.203 | WRC | 9.7 |
| 08013000 | 0.144 | 0.151 | 0.156 | WRC | 8.3 |
| 08013500 | 0.181 | 0.178 | 0.182 | MIX | 2.2 |
| 08014500 | 0.664 | 0.640 | 0.613 | MDM | 8.3 |
| *08014500 | 0.086 | 0.106 | 0.110 | WRC | 27.9 |
| 08015500 | 0.336 | 0.317 | 0.319 | MIX | 6.0 |
| *08015500 | 0.121 | 0.126 | 0.127 | WRC | 5.0 |
| 08014000 | 0.278 | 0.302 | 0.292 | WRC | 8.6 |
| 08025500 | 0.328 | 0.383 | 0.293 | MDM | 30.7 |
| 08028000 | 0.252 | 0.288 | 0.214 | MDM | 34.6 |
| 02490105 | 0.197 | 0.218 | 0.188 | MDM | 16.0 |
| 07375222 | 0.179 | 0.137 | 0.095 | MDM | 88.4 |
| 07380160 | 0.075 | 0.069 | 0.064 | MDM | 17.2 |
| 07375170 | 0.145 | 0.164 | 0.155 | WRC | 13.1 |
| 07377300 | 0.103 | 0.120 | 0.120 | WRC | 16.5 |
| 07376600 | 0.046 | 0.048 | 0.048 | WRC | 4.3 |
| 07375480 | 0.168 | 0.218 | 0.218 | WRC | 29.8 |
| 02491700 | 0.153 | 0.219 | 0.229 | WRC | 49.7 |
| 02491350 | 0.159 | 0.198 | 0.179 | WRC | 24.5 |
| 07375800 | 0.259 | 0.349 | 0.326 | WRC | 34.7 |
| 07375307 | 0.247 | 0.283 | 0.232 | MDM | 22.0 |
| 07373500 | 0.101 | 0.109 | 0.102 | WRC | 7.9 |
| 07364300 | 0.263 | 0.282 | 0.281 | WRC | 7.2 |
| 07369500 | 0.030 | 0.030 | 0.028 | MDM | 7.1 |
| 07386500 | 0.132 | 0.108 | 0.104 | MDM | 26.9 |
| 08011800 | 0.119 | 0.141 | 0.146 | WRC | 22.7 |
| 08014200 | 0.412 | 0.410 | 0.400 | MDM | 3.0 |
| 07353500 | 0.164 | 0.234 | 0.214 | WRC | 42.7 |
| 07372500 | 0.604 | 0.643 | 0.589 | MDM | 9.2 |
| *07372500 | 0.196 | 0.205 | 0.199 | WRC | 4.6 |
| 07370750 | 0.233 | 0.274 | 0.258 | WRC | 17.6 |
| 07372110 | 0.378 | 0.451 | 0.346 | MDM | 30.3 |
| 07372000 | 0.089 | 0.090 | 0.089 | WRC/MDM | 1.1 |
| 07370500 | 0.149 | 0.141 | 0.147 | MIX | 5.7 |
| 07370000 | 0.090 | 0.090 | 0.086 | MDM | 4.7 |
| 07367250 | 0.430 | 0.453 | 0.357 | MDM | 26.9 |
| 07366403 | 0.263 | 0.326 | 0.265 | WRC | 24.0 |
| 07366420 | 0.488 | 0.537 | 0.502 | WRC | 10.0 |
| 07365000 | 0.180 | 0.191 | 0.189 | WRC | 6.1 |

TABLE 2 (*continued on next page*)

TABLE 2 (*continued*)

| Gage Station | SRMSD | | | Method(s) with Min. SRMSD | Max. Diff. % |
|---|---|---|---|---|---|
| | WRC | MIX | MDM | | |
| 07364870 | 0.223 | 0.193 | 0.195 | MIX | 15.5 |
| 07365500 | 0.596 | 0.651 | 0.593 | MDM | 9.8 |
| *07365500 | 0.157 | 0.181 | 0.174 | WRC | 15.3 |
| 07366000 | 0.485 | 0.471 | 0.453 | MDM | 7.1 |
| 07364700 | 0.706 | 0.732 | 0.566 | MDM | 29.3 |
| 08016600 | 0.166 | 0.157 | 0.145 | MDM | 14.5 |
| 08028700 | 0.268 | 0.279 | 0.276 | WRC | 4.1 |
| *08028700 | 0.115 | 0.116 | 0.120 | WRC | 4.4 |
| 08014600 | 0.252 | 0.284 | 0.256 | WRC | 12.7 |
| 08013800 | 0.161 | 0.173 | 0.171 | WRC | 7.5 |
| 08013610 | 0.084 | 0.081 | 0.064 | MDM | 31.3 |
| 07354000 | 0.063 | 0.066 | 0.063 | WRC/MDM | 4.8 |
| 07353990 | 0.216 | 0.259 | 0.207 | MDM | 25.1 |
| 08016800 | 0.308 | 0.300 | 0.298 | MDM | 3.4 |
| 08016400 | 0.139 | 0.164 | 0.160 | WRC | 18.0 |
| 08015000 | 0.178 | 0.220 | 0.201 | WRC | 23.6 |
| 07352500 | 0.159 | 0.143 | 0.125 | MDM | 27.2 |
| 02490000 | 0.235 | 0.290 | 0.295 | WRC | 25.5 |
| 07348725 | 0.157 | 0.136 | 0.128 | MDM | 22.7 |
| 07348800 | 0.205 | 0.245 | 0.235 | WRC | 19.5 |
| 07347000 | 0.178 | 0.174 | 0.177 | MIX | 2.3 |
| *07347000 | 0.078 | 0.072 | 0.075 | MIX | 8.3 |
| 07362100 | 0.298 | 0.311 | 0.306 | WRC | 4.4 |
| 07364190 | 0.089 | 0.085 | 0.081 | MDM | 9.9 |
| 07365800 | 0.915 | 0.937 | 0.802 | MDM | 16.8 |
| *07365800 | 0.641 | 0.623 | 0.612 | MDM | 4.7 |
| 07373550 | 0.067 | 0.068 | 0.071 | WRC | 6.0 |
| 08014800 | 0.145 | 0.173 | 0.178 | WRC | 22.8 |
| 08025850 | 0.334 | 0.384 | 0.334 | WRC/MDM | 15.0 |
| 08024060 | 0.092 | 0.088 | 0.089 | MIX | 4.5 |
| 08023000 | 0.130 | 0.158 | 0.161 | WRC | 23.8 |
| 07351700 | 0.976 | 0.967 | 0.895 | MDM | 9.1 |
| *07351700 | 0.146 | 0.107 | 0.102 | MDM | 43.1 |
| 07368500 | 0.054 | 0.056 | 0.057 | WRC | 5.6 |
| 07364500 | 0.108 | 0.095 | 0.083 | MDM | 30.1 |
| 02492360 | 0.104 | 0.122 | 0.114 | WRC | 17.3 |
| 08031000 | 0.147 | 0.157 | 0.151 | WRC | 6.8 |
| 08030000 | 0.155 | 0.157 | 0.160 | WRC | 3.2 |
| 08029500 | 0.337 | 0.414 | 0.350 | WRC | 22.8 |

*Run without the outliers.

the skewness of the data samples. Tables 2 and 3 indicate that for the three samples that exhibit a negative skew coefficient, MDM is the preferred estimator by a significant amount in two of the three cases where MIX and WRC performed comparably. For the 19 cases that exhibit skew coefficients in the range 0 to 1.0, MDM is superior in 7 cases (by SRMSD) by a significant margin; WRC and MIX are preferred by a significant amount in one case each; whereas the methods perform about equally well in the other cases. These results are approximately mirrored in the SMAD cases as well. Most of the data samples in this study exhibit skew coefficients that lie in the range 1 to 3. Of the 68 samples in this moderate range of skewness, over two-thirds were better fitted by the WRC method. Of the samples that were better fitted by the WRC method, two-thirds exhibited maximum SRMSD and SMAD difference $\geq$ 10 percent. On the other hand, of the 13 samples that exhibited skew coefficients greater than 3.0, MDM and MIX were the preferred estimators in terms of SRMSD in 11 cases. However, in these cases the SRMSD differences were less than 10 percent in all but two instances. Thus, it appears that the WRC method is superior in descriptive ability when the data samples exhibit moderate skewness ($1 \leq \gamma \leq 3$), whereas for the samples of small skewness ($\gamma < 1.0$), MDM or MIX may be superior. This result is particularly evident in the small number of cases that exhibited negative skewness in the raw data. For the cases of large skewness ($\gamma > 3.0$) there was no significant difference in the performance of the methods. However, the performance of

TABLE 3 SMAD TEST RESULTS FOR THE THREE LP3 FITTING METHODS

| Gage Station | SMAD WRC | SMAD MIX | SMAD MDM | Method(s) with Min. SMAD | Max. Diff. % |
|---|---|---|---|---|---|
| 02491500 | 0.101 | 0.102 | 0.107 | WRC | 5.9 |
| 02492000 | 0.109 | 0.109 | 0.109 | WRC/MIX/MDM | 0.0 |
| 07344450 | 0.150 | 0.159 | 0.190 | WRC | 26.7 |
| 07348700 | 0.116 | 0.122 | 0.128 | WRC | 10.3 |
| 07349500 | 0.067 | 0.067 | 0.069 | WRC/MIX | 3.0 |
| 07351500 | 0.086 | 0.062 | 0.063 | MIX | 38.7 |
| 07352000 | 0.067 | 0.081 | 0.075 | WRC | 20.9 |
| 07351000 | 0.064 | 0.065 | 0.069 | WRC | 7.8 |
| 07366200 | 0.190 | 0.186 | 0.217 | MIX | 16.7 |
| 07371500 | 0.090 | 0.078 | 0.081 | MIX | 15.4 |
| 07372200 | 0.087 | 0.090 | 0.089 | WRC | 3.4 |
| 07373000 | 0.112 | 0.119 | 0.099 | MDM | 20.2 |
| 07375000 | 0.091 | 0.094 | 0.083 | MDM | 13.2 |
| 07375500 | 0.084 | 0.086 | 0.084 | WRC/MDM | 2.4 |
| 07376000 | 0.067 | 0.078 | 0.081 | WRC | 20.9 |
| 07376500 | 0.050 | 0.055 | 0.059 | WRC | 18.0 |
| 07377000 | 0.103 | 0.086 | 0.080 | MDM | 28.8 |
| 07377500 | 0.121 | 0.096 | 0.072 | MDM | 68.1 |
| 07378000 | 0.052 | 0.052 | 0.053 | WRC/MIX | 1.9 |
| 07378500 | 0.064 | 0.069 | 0.067 | WRC | 7.8 |
| 07381800 | 0.073 | 0.090 | 0.085 | WRC | 23.3 |
| 07382000 | 0.211 | 0.281 | 0.464 | WRC | 119.9 |
| *07382000 | 0.085 | 0.092 | 0.121 | WRC | 42.4 |
| 08010000 | 0.057 | 0.044 | 0.044 | MIX/MDM | 29.5 |
| 08012000 | 0.098 | 0.115 | 0.159 | WRC | 62.2 |
| *08012000 | 0.079 | 0.086 | 0.117 | WRC | 48.1 |
| 08013000 | 0.092 | 0.093 | 0.094 | WRC | 2.2 |
| 08013500 | 0.079 | 0.081 | 0.080 | WRC | 2.5 |
| 08014500 | 0.156 | 0.154 | 0.210 | MIX | 36.4 |
| *08014500 | 0.057 | 0.070 | 0.073 | WRC | 28.1 |
| 08015500 | 0.117 | 0.122 | 0.122 | WRC | 4.3 |
| *08015500 | 0.069 | 0.069 | 0.069 | WRC/MIX/MDM | 0.0 |
| 08014000 | 0.136 | 0.146 | 0.160 | WRC | 17.6 |
| 08025500 | 0.184 | 0.246 | 0.230 | WRC | 33.7 |
| 08028000 | 0.154 | 0.168 | 0.132 | MDM | 27.3 |
| 02490105 | 0.096 | 0.122 | 0.128 | WRC | 33.3 |
| 07375222 | 0.134 | 0.116 | 0.083 | MDM | 61.4 |
| 07380160 | 0.067 | 0.059 | 0.049 | MDM | 36.7 |
| 07375170 | 0.111 | 0.123 | 0.115 | WRC | 10.8 |
| 07377300 | 0.080 | 0.085 | 0.080 | WRC/MDM | 6.3 |
| 07376600 | 0.041 | 0.042 | 0.039 | MDM | 7.7 |
| 07375480 | 0.112 | 0.116 | 0.110 | MDM | 5.5 |
| 02491700 | 0.126 | 0.153 | 0.155 | WRC | 23.0 |
| 02491350 | 0.125 | 0.128 | 0.104 | MDM | 23.1 |
| 07375800 | 0.121 | 0.136 | 0.116 | MDM | 17.2 |
| 07375307 | 0.138 | 0.140 | 0.123 | MDM | 13.8 |
| 07373500 | 0.071 | 0.076 | 0.070 | MDM | 8.6 |
| 07364300 | 0.119 | 0.125 | 0.121 | WRC | 5.0 |
| 07369500 | 0.025 | 0.026 | 0.023 | MDM | 13.0 |
| 07386500 | 0.075 | 0.068 | 0.062 | MDM | 21.0 |
| 08011800 | 0.095 | 0.097 | 0.096 | WRC | 2.1 |
| 08014200 | 0.119 | 0.119 | 0.133 | WRC/MIX | 11.8 |
| 07353500 | 0.125 | 0.136 | 0.116 | MDM | 17.2 |
| 07372500 | 0.180 | 0.203 | 0.268 | WRC | 48.9 |
| *07372500 | 0.111 | 0.120 | 0.132 | WRC | 18.9 |
| 07370750 | 0.095 | 0.114 | 0.135 | WRC | 42.1 |
| 07372110 | 0.192 | 0.227 | 0.196 | WRC | 18.2 |
| 07372000 | 0.055 | 0.053 | 0.055 | MIX | 3.8 |
| 07370500 | 0.097 | 0.091 | 0.090 | MDM | 7.8 |
| 07370000 | 0.066 | 0.066 | 0.068 | WRC/MIX | 3.0 |
| 07367250 | 0.202 | 0.262 | 0.268 | WRC | 32.7 |
| 07366403 | 0.140 | 0.173 | 0.154 | WRC | 23.6 |
| 07366420 | 0.181 | 0.188 | 0.200 | WRC | 10.5 |
| 07365000 | 0.093 | 0.095 | 0.100 | WRC | 7.5 |

TABLE 3 (*continued on next page*)

TABLE 3 (*continued*)

| Gage Station | SMAD WRC | SMAD MIX | SMAD MDM | Method(s) with Min. SMAD | Max. Diff. % |
|---|---|---|---|---|---|
| 07364870 | 0.151 | 0.131 | 0.121 | MDM | 24.8 |
| 07365500 | 0.188 | 0.196 | 0.227 | WRC | 17.2 |
| *07365500 | 0.124 | 0.130 | 0.118 | MDM | 10.2 |
| 07366000 | 0.186 | 0.181 | 0.231 | MIX | 27.6 |
| 07364700 | 0.294 | 0.369 | 0.366 | WRC | 25.5 |
| 08016600 | 0.114 | 0.120 | 0.120 | WRC | 5.0 |
| 08028700 | 0.116 | 0.118 | 0.136 | WRC | 17.2 |
| *08028700 | 0.080 | 0.077 | 0.082 | MIX | 6.5 |
| 08014600 | 0.160 | 0.181 | 0.188 | WRC | 17.5 |
| 08013800 | 0.085 | 0.090 | 0.092 | WRC | 8.2 |
| 08013610 | 0.072 | 0.071 | 0.053 | MDM | 35.8 |
| 07354000 | 0.051 | 0.053 | 0.053 | WRC | 3.9 |
| 07353990 | 0.110 | 0.142 | 0.141 | WRC | 29.1 |
| 08016800 | 0.145 | 0.140 | 0.155 | MIX | 10.7 |
| 08016400 | 0.080 | 0.093 | 0.102 | WRC | 27.5 |
| 08015000 | 0.109 | 0.130 | 0.114 | WRC | 19.3 |
| 07352500 | 0.094 | 0.092 | 0.097 | MIX | 5.4 |
| 02490000 | 0.159 | 0.175 | 0.177 | WRC | 11.3 |
| 07348725 | 0.124 | 0.113 | 0.105 | MDM | 18.1 |
| 07348800 | 0.084 | 0.100 | 0.091 | WRC | 19.0 |
| 07347000 | 0.081 | 0.076 | 0.081 | MIX | 6.6 |
| *07347000 | 0.054 | 0.047 | 0.049 | MIX | 14.9 |
| 07362100 | 0.097 | 0.100 | 0.111 | WRC | 14.4 |
| 07364190 | 0.067 | 0.065 | 0.063 | MDM | 6.3 |
| 07365800 | 0.344 | 0.353 | 0.427 | WRC | 24.1 |
| *07365800 | 0.246 | 0.254 | 0.264 | WRC | 7.3 |
| 07373550 | 0.053 | 0.053 | 0.055 | WRC/MIX | 3.8 |
| 08014800 | 0.109 | 0.119 | 0.115 | WRC | 9.2 |
| 08025850 | 0.172 | 0.199 | 0.210 | WRC | 22.1 |
| 08024060 | 0.079 | 0.073 | 0.073 | MIX/MDM | 8.2 |
| 08023000 | 0.098 | 0.105 | 0.099 | WRC | 7.1 |
| 07351700 | 0.355 | 0.355 | 0.446 | WRC/MIX | 25.6 |
| *07351700 | 0.124 | 0.088 | 0.076 | MDM | 63.2 |
| 07368500 | 0.038 | 0.039 | 0.039 | WRC | 2.6 |
| 07364500 | 0.068 | 0.062 | 0.054 | MDM | 25.9 |
| 02492360 | 0.077 | 0.091 | 0.083 | WRC | 18.2 |
| 08031000 | 0.089 | 0.093 | 0.103 | WRC | 15.7 |
| 08030000 | 0.087 | 0.086 | 0.093 | MIX | 8.1 |
| 08029500 | 0.162 | 0.197 | 0.208 | WRC | 28.4 |

*Run without the outliers.

all three methods decreased significantly as the skew coefficient increased. The average SRMSD for the MDM of the five samples with the smallest skew coefficients is 0.074, whereas the average SRMSD for the MDM of the five samples with the largest skew coefficients is 0.739. This result represents a deterioration in SRMSD performance of 892 percent. The MDM resulted in the better fit in all 10 of these extreme cases.

## CONCLUSION

The results of this study demonstrate that in many cases there is a significant difference, depending on sample skewness, between the descriptive capability of these three moment-based methods. However, no method demonstrated clear superiority across all samples. For samples that exhibit skew coefficients greater than 1.0, the WRC method performs comparatively well in terms of both performance indices. For samples that exhibit skew coefficients of less than 1.0, the WRC method is clearly inferior to MDM and MIX, on the basis of the limited number of samples in this range. Previous Monte Carlo studies (*7,8*) that compared the relative predictive ability of these methods were based on samples generated from known populations and generally concluded that WRC did not perform as well as MDM and MIX in this regard. However, the results of the Monte Carlo studies may not translate to the real-world situations wherein the populations are unknown.

TABLE 4 COMPARISON OF ROBUSTNESS

| Method | Min | Average | Max |
|---|---|---|---|
| | | SRMSD | |
| WRC | .030 | .238 (.196) | 1.096 (.706) |
| MIX | .030 | .252 (.209) | 1.150 (.732) |
| MDM | .028 | .233 (.195) | 1.004 (.612) |
| | | SMAD | |
| WRC | .025 | .114 (.105) | .355 (.294) |
| MIX | .026 | .121 (.111) | .369 (.369) |
| MDM | .023 | .126 (.111) | .464 (.366) |

Note: Values in parenthesis denote performance indices without the outliers.

The results may be of particular significance to engineers working in the area of highway drainage design. These structures are frequently designed for small recurrence intervals. The results demonstrate that for data with skew coefficients greater than 1.0, but particularly in the range $1 \leq \gamma \leq 3$, the WRC method possesses superior interpretive ability. Thus, it appears that this method may continue to be used confidently by engineers engaged in the design of small drainage structures.

## ACKNOWLEDGMENT

This study was supported by the Louisiana Transportation Research Center. Stream data were provided by the U.S. Geological Survey. The authors wish to express their thanks to Glenn Chutz for his assistance in this work.

## REFERENCES


1. B. Bobee. The Log Pearson Type 3 Distribution and Its Application in Hydrology. *Water Resources Research*, Vol. 11, No. 5, 1975, pp. 681–689.
2. B. Bobee and R. Robitaille. Correction of Bias in the Estimation of the Coefficient of Skewness. *Water Resources Research*, Vol. 11, No. 6, 1975, pp. 851–854.
3. R. Condie. The Log Pearson Type 3 Distribution: The T-Year Event and Its Asymptotic Standard Error by Maximum Likelihood Theory. *Water Resources Research*, Vol. 13, No. 6, 1977, pp. 987–991.
4. D. V. Rao. Log Pearson Type 3 Distribution: Method of Mixed Moments. *Journal of Hydraulics Division, ASCE*, Vol. 106, No. 6, 1980, pp. 999–1019.
5. *Guidelines for Determining Flood Flow Frequency*. U.S. Water Resources Council, Bulletin 17B, Washington, D.C., 1982.
6. V. P. Singh and K. Singh. Parameter Estimation for Log-Pearson Type III Distribution by POME. *Journal of Hydraulic Engineering, ASCE*, Vol. 114, No. 1, 1988, pp. 112–122.
7. K. Arora and V. P. Singh. A Comparative Evaluation of the Estimators of Log Pearson Type 3 Distribution. *Journal of Hydrology*, Vol. 105, 1989, pp. 19–37.
8. F. Ashkar and B. Bobee. The Generalized Method of Moments as Applied to Problems of Flood Frequency Analysis: Some Practical Results for the Log-Pearson Type 3 Distribution. *Journal of Hydrology*, Vol. 90, 1987, pp. 199–217.
9. B. Bobee and F. Ashkar. Generalized Method of Moments Applied to LP3 Distribution. *Journal of Hydraulic Engineering, ASCE*, Vol. 114, No. 8, 1988, pp. 899–909.
10. K. Arora and V. P. Singh. On the Method of Maximum Likelihood Estimation for the Log-Pearson Type 3 Distribution. *Journal of Stochastic Hydrology and Hydraulics*, Vol. 2, No. 2, 1988, pp. 155–160.
11. H. N. Phien and V. Nguyen. Discussion on Derivation of the Pearson (PT) III Distribution by Using the Principle of Maximum Entropy (POME). *Journal of Hydrology*, Vol. 90, 1987, pp. 351–357.
12. J. R. Wallis, N. C. Matalas, and J. R. Slack. Just a Moment! *Water Resources Research*, Vol. 10, No. 2, 1974, pp. 211–219.
13. W. Kirby. Algebraic Boundedness of Sample Statistics. *Water Resources Research*, Vol. 10, No. 2, 1974, pp. 220–222.
14. N. C. Matalas, J. R. Slack, and J. R. Wallis. Regional Skew in Search of a Parent. *Water Resources Research*, Vol. 11, No. 6, 1975, pp. 815–826.
15. J. R. Wallis and E. F. Wood. Relative Accuracy of Log Pearson III Procedures. *Journal of Hydraulic Division, ASCE*, Vol. 111, No. 7, 1985, pp. 1043–1056.
16. C. Cunnane. Review of Statistical Models for Flood Frequency Estimation. In *Hydrologic Frequency Modeling*, V. P. Singh, ed., Reidel, Dordrecht, Holland, 1987, pp. 49–95.
17. F. E. Grubbs. Procedures for Detecting Outlying Observations in Samples. *Technometrics*, Vol. 11, No. 1, 1969, pp. 1–7.
18. G. Kuczera. Robust Flood Frequency Models. *Water Resources Research*, Vol. 18, No. 2, 1982, pp. 315–324.


*Publication of this paper sponsored by Committee on Hydrology, Hydraulics, and Water Quality.*