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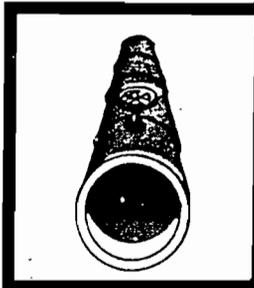
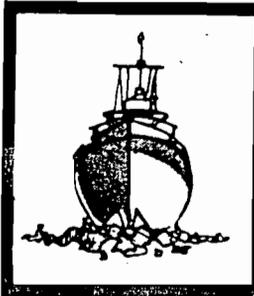
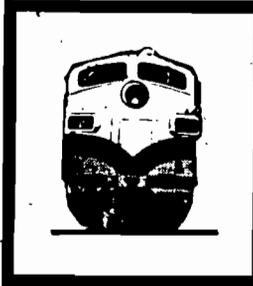
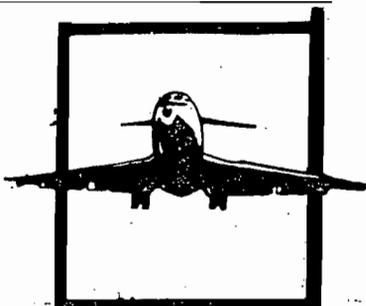
WASHINGTON, D.C. 20594

## HIGHWAY ACCIDENT REPORT

COLLAPSE OF  
NEW YORK THRUWAY (I-90) BRIDGE  
OVER THE SCHOHARIE CREEK,  
NEAR AMSTERDAM, NEW YORK  
APRIL 5, 1987

NTSB/HAR-88/02

UNITED STATES GOVERNMENT



## ERRATA

**THESE CORRECTIONS SHOULD BE MADE  
TO THE PREVIOUSLY PUBLISHED REPORT  
IDENTIFIED AS FOLLOWS**

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NEAR AMSTERDAM, NEW YORK  
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**NTSB/HAR-88/02 (PB88-916202)**

Page 46, paragraph 1, lines 4-7

#### Change

For bridge inspections, the lines of command do not follow the formal organization structure. In actuality, the assistant division engineers (bridges) report directly to the assistant superintendent of maintenance (bridges).

#### To

Under the formal lines of command the assistant division engineer (bridges) reported directly to the division engineer. However, the assistant division engineer (bridges) often consulted with the assistant superintendent of maintenance (bridges) on bridge inspection matters, and from a practical standpoint accepted direction and guidance from the superintendent on such matters.

Page 119, paragraph 5, line 4 and 5

#### Change

For example, the NYSTA assistant superintendent of maintenance (bridges), the bridge inspector's supervisor, said . . . .

#### To

For example, the NYSTA assistant superintendent of maintenance (bridges) said . . . .

TECHNICAL REPORT DOCUMENTATION PAGE

1. Report No. NTSB/HAR-88/02		2. Government Accession No. PB88-916202		3. Recipient's Catalog No.	
4. Title and Subtitle Highway Accident Report-- Collapse of New York Thruway (I-90) Bridge Over the Schoharie Creek, Near Amsterdam, New York, April 5, 1987				5. Report Date April 29, 1988	
				6. Performing Organization Code	
7. Author(s)				8. Performing Organization Report No.	
9. Performing Organization Name and Address National Transportation Safety Board Bureau of Accident Investigation Washington, D.C. 20594				10. Work Unit No. 4656A	
				11. Contract or Grant No.	
12. Sponsoring Agency Name and Address  NATIONAL TRANSPORTATION SAFETY BOARD Washington, D. C. 20594				13. Type of Report and Period Covered  Highway Accident Report April 5, 1987	
				14. Sponsoring Agency Code	
15. Supplementary Notes					
16. Abstract  This accident investigation delves into the causes of the collapse of a 5-span, 540-foot-long highway bridge over the Schoharie Creek in Montgomery County near Amsterdam, New York, on April 5, 1987. The report discusses the design and construction of the bridge, the intensity of previous floods, the vulnerability of the soil to scour and the bridge's dependency on riprap protection, and the suitability of spread footings in streambeds subject to high velocity flows. The report also discusses the maintenance and inspection history of the bridge. The report discusses deficiencies uncovered in the bridge inspection programs of the New York State Thruway Authority (owner of the bridge) and the New York State Department of Transportation, and the oversight of their programs by the Federal Highway Administration.  The National Transportation Safety Board determines that the probable cause of the collapse of the Schoharie Creek Bridge was the failure of the New York State Thruway Authority to					
17. Key Words Highway bridge, scour, piles, riprap, nonredundant structure, New York State Thruway Authority, Schoharie Creek, bridge maintenance and inspection, hydraulic studies, and modeling erosion, New York State Department of Transportation, Federal Highway Administration, U.S. DOT Inspector General, National Bridge Inspection Standards				18. Distribution Statement This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classification (of this report) UNCLASSIFIED		20. Security Classification (of this page) UNCLASSIFIED		21. No. of Pages 168	22. Price

to maintain adequate riprap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the accident were ambiguous plans and specifications used for construction of the bridge, an inadequate NYSTA bridge inspection program, and inadequate oversight by the New York State Department of Transportation and the Federal Highway Administration. Contributing to the severity of the accident was the lack of structural redundancy in the bridge.

Recommendations are proposed to revise existing guidelines for design, maintenance, and inspection of bridges. In addition, it is recommended that the U.S. Department of Transportation Inspector General periodically review the FHWA bridge inspection audit program for compliance with the National Bridge Inspection Standards.

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## EXECUTIVE SUMMARY

On April 5, 1987, two spans of the New York State Thruway (I-90) bridge over the Schoharie Creek fell about 80 feet into a rain-swollen creek after pier 3, which partially supported the spans, collapsed. Ninety minutes after the initial collapse, pier 2 and a third span collapsed. Four passenger cars and one tractor-semitrailer plunged into the creek, and 10 persons were fatally injured.

The National Transportation Safety Board determines that the probable cause of the collapse of the Schoharie Creek Bridge was the failure of the New York State Thruway Authority to maintain adequate riprap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the accident were ambiguous plans and specifications used for construction of the bridge, an inadequate NYSTA bridge inspection program, and inadequate oversight by the New York State Department of Transportation and the Federal Highway Administration. Contributing to the severity of the accident was the lack of structural redundancy in the bridge.

The primary safety issues raised by this accident are the adequacy of the bridge design and construction, the adequacy of the inspection and maintenance practices for this bridge, and the adequacy of the Federal/State oversight of private independent authorities. The report also evaluates the adequacy of existing Federal Highway Administration/American Association of State Highway and Transportation Officials requirements for underwater inspections of scour.

Recommendations have been made to the NYSTA, Federal Highway Administration, and the American Association of State Highway and Transportation Officials to correct deficiencies uncovered.

TRANSPORTATION SAFETY BOARD  
WASHINGTON, D.C. 20594

HIGHWAY ACCIDENT REPORT

Adopted: April 26, 1988

COLLAPSE OF NEW YORK THRUWAY  
(I-90) BRIDGE OVER THE SCHOHARIE CREEK  
NEAR AMSTERDAM, NEW YORK  
APRIL 5, 1987

INVESTIGATION

The Accident

About 10:44 a.m. eastern daylight time 1/ on April 5, 1987, traffic was light to moderate on the New York State Thruway Authority (NYSTA) Bridge (Schoharie Creek Bridge) over the Schoharie Creek. The four-lane, 540-foot-long bridge crossed the Schoharie Creek in Montgomery County, about 6 miles west of Amsterdam, New York. It was raining lightly, but, according to witnesses, visibility through the rain was good. Water levels in the Schoharie Creek had been rising steadily since the afternoon of April 4. The National Weather Service (NWS) issued warnings throughout the morning advising residents and motorists of flooding in the low lying areas adjacent to the creek. A state trooper who crossed the bridge about 10:40 a.m. did not observe any unusual discrepancies.

About 10:45 a.m., pier 3 collapsed and the third and fourth spans of the Schoharie Creek Bridge 2/ fell about 80 feet into the rain-swollen creek. (See figures 1 and 2.) Four persons who were monitoring the high water conditions at the Route 5S Bridge (about 2,000 feet downstream from the Schoharie Creek Bridge) observed the collapse. Two of these persons, who were local volunteer firemen, stated that they initially heard a loud noise that sounded like "an explosion or thunder." When they looked up, they saw two sections of the bridge fall. They stated that both sections fell together rapidly along with two eastbound vehicles (a tractor-semitrailer and a passenger car), which were traveling across the falling bridge sections. Shortly afterwards, two more eastbound cars drove into the void in the bridge and plunged into the creek. About a second later, a westbound car also drove into the void and sank immediately into the fast moving water. Shortly afterwards, one witness observed bundles of paper products and what appeared to be oil on top of the water.

1/ On April 5, 1987, the time changed from eastern standard to eastern daylight time.

2/ A bridge is defined as "a structure including supports, erected over a depression or an obstruction, such as water, a highway, or railway having a track or passageway for carrying traffic or other moving loads and having an opening measured along the centerline of the roadway of more than 20 feet." American Association of State Highway and Transportation Officials Definitions, Washington, D.C., 1988, p. 2.

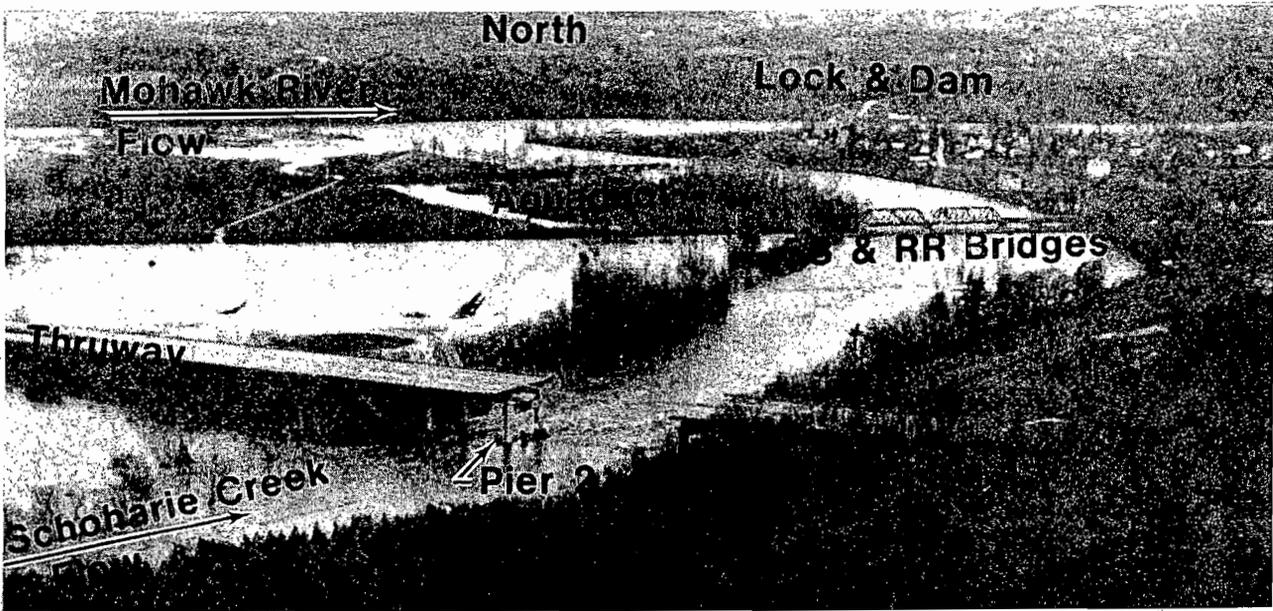


Figure 1.--Flood of 1987 after the initial collapse looking north.

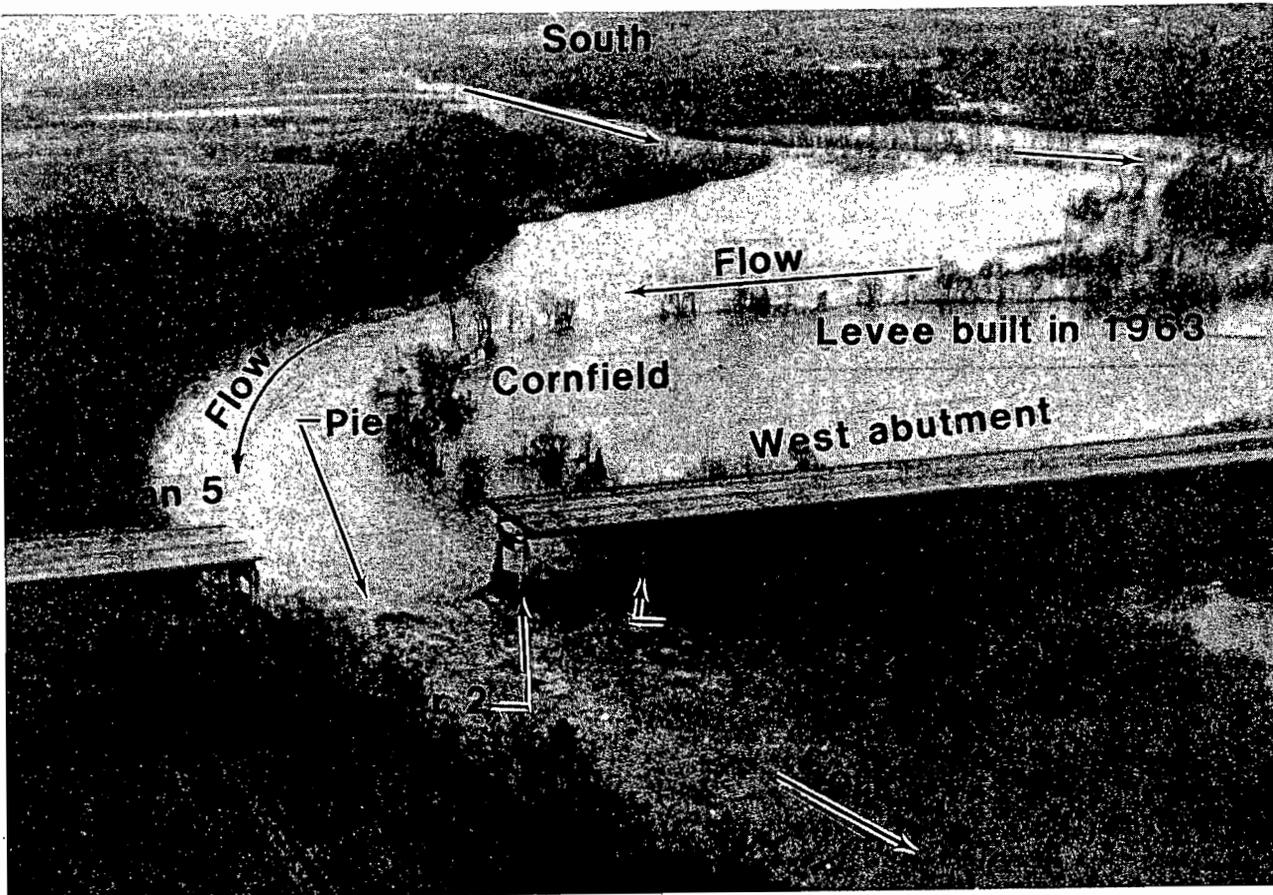


Figure 2.--Flood of 1987 after the initial collapse looking south.

Other motorists traveling behind the fallen vehicles recognized the impending danger and stopped their vehicles. After stopping, several motorists got out of their cars and began to "wave down" other motorists to prevent them from traveling into the void on the bridge.

Four passenger cars and one tractor-semitrailer with a total of ten occupants plunged into the creek. Nine bodies were recovered, but one person is still missing and is presumed to be dead.

Video films and a high speed sequence of pictures obtained from the news media showed that pier 2 and span 2 collapsed into the creek about 90 minutes after the collapse of pier 3. No vehicles or persons were on the span, which fell into the water suddenly with little warning. About 1:15 p.m., pier 1 moved, and dislodged span 1 from both expansion bearings. Sometime during the collapse sequence, span 5 slipped off its south expansion bearing but remained in place. Spans 1 and 5 and piers 1 and 4 remained standing.

#### Emergency Response

One of the local firemen standing on the Route 5S Bridge immediately notified the Fort Hunter Volunteer Fire Department of the collapse. After requesting assistance, he and the other firemen proceeded to the bridge site to assist in the rescue operation. The Fort Hunter Volunteer Fire Department relayed the message to the Montgomery County Sheriff's Department, which then notified the NYSTA Fultonville Interchange toll collector.

At 10:48 a.m., the toll collector advised the NYSTA Albany dispatcher of the report that the Thruway bridge over the Schoharie Creek had collapsed, and that a truck and several cars had gone into the creek. Two New York State Police Troopers were immediately dispatched to the bridge site to verify the report. Both troopers arrived at the bridge site about 10:50 a.m., one on the eastbound approach and the other on the westbound approach to the bridge. The troopers confirmed the initial report and then removed all motorists and passersby from the portions of the bridge still standing.

At about 11:05 a.m., a State Police captain arrived and established a command center. He began directing the traffic control and the search and rescue operations. After having the Thruway closed at the toll facilities on both sides of the bridge, he requested tow trucks to retrieve accident vehicles, and NYSTA maintenance and engineering personnel to provide assistance.

The State Police captain also initiated a shoreline search for survivors. The initial search did not uncover any accident victims or vehicles. An aviation unit and divers were requested to participate in the search for survivors. The search area along the Schoharie Creek was divided between the Montgomery County Sheriff's Department and the State Police. The State

Police searched downstream (north) from the bridge site to the 5S Bridge (about 2,000 feet) and the Sheriff's Department continued the search downstream of the 5S bridge north toward the mouth of the Mohawk River for an additional 3,500 feet.

The search and rescue efforts continued from April 5 through April 12, 1987. The State Police search group consisted of 14 uniform troopers, 17 divers, 5 dog handlers and dogs, and 8 support personnel; the Montgomery County Sheriff's Department group consisted of 5 divers and 4 officers. The first body was found on April 5, and the ninth body was found on April 26, 1987. One body is still missing.

### Injuries to Persons 3/

<u>Fatally Injured</u>	<u>Occupants</u>		<u>Total</u>
	<u>Cars</u>	<u>Trucks</u>	
Unknown (AIS-9) 4/ 5/	1	1	2
Maximum Injury-Virtually Unsurvivable (AIS-6)	5	-	5
Critical (AIS-5)	1	-	1
Severe (AIS-4)	<u>1</u>	<u>-</u>	<u>1</u>
Total	8	1	9

### Vehicle and Occupant Information

The four passenger cars and the tractor-semitrailer that fell into the creek were completely demolished. The roofs of the passenger cars were crushed to the tops of the dashboards and seats. The tractor and semitrailer separated during the collision sequence and came to rest about 300 feet apart downstream. The fiberglass body of the tractor was shattered into several small pieces and the box type semitrailer was ripped open on the left side.

All of the accident vehicles sustained substantial water damage and were swept downstream from the bridge site before coming to rest. One vehicle traveled about 4,700 feet from the bridge site. A postcrash examination of the accident vehicles did not reveal any preexisting mechanical defects that may have contributed to the accident.

3/ No AIS injury classification could be made concerning injuries sustained by the one occupant who was presumed to have been fatally injured but has not been recovered.

4/ The AIS (abbreviated injury scale) was developed by the American Association for Automotive Medicine.

5/ The cause of death of both persons coded AIS-9 was drowning. Because of the unique circumstances of this accident, these persons sustained traumatic injuries. Although these injuries were well documented, it could not be determined which injuries were antemortem and which were postmortem.

Three of the cars were occupied by a driver and one passenger, one car was carrying a driver and two passengers, and the tractor-semitrailer was occupied only by a driver. (See appendix B for additional vehicle and occupant information.)

### Highway Information

The 559-mile New York State Thruway is a toll road that extends north from New York City to Albany, west to Buffalo, and then southwest to the Pennsylvania Border. In 1942, a New York State bill authorized planning, design, and construction of the cross-State superhighway. The first 4-mile segment opened in 1948. In 1950, a special committee was established to develop a self-supporting base to fund the construction and operation of the superhighway. On March 21, 1950, the Governor of New York signed legislation authorizing the creation of the NYSTA. The NYSTA was formally established in 1951 and staffed. Between 1948 and October 1954, an additional 149 miles of the Thruway were opened. On October 26, 1954, the Governor of New York participated in a motorcade that opened 183 miles of the Thruway, including the Schoharie Creek Bridge. This brought the total mileage of the Thruway to 336 miles. <sup>6/</sup>

In the vicinity of the bridge collapse, the Thruway runs generally east-west adjacent to the Mohawk River and is designated as Interstate (I) 90. The roadway approaching and crossing the Schoharie Creek is a four-lane divided highway. The posted speed limit at the time of the collapse was 55 mph.

In 1986, the Average Daily Traffic (ADT) over the Schoharie Creek Bridge was 15,519 vehicles per day. In 1986, the Thruway fatality rate was 0.89 per 100 million miles traveled. In comparison, the nationwide fatality rate for 1986 was 1.09 per 100 million vehicle miles traveled on interstate roads.

### Meteorological Information

In the week before the bridge collapse, heavy rains fell in the headwater area upstream of the bridge site. The National Weather Service (NWS) recorded approximately 7.30 inches of rain at Tannersville, New York, about 52 miles southeast of the bridge, during the weekend of April 3, 1987. Heavy rains had fallen in the same area on March 30 and 31, saturating the ground. Finally, the snowmelt in the Catskill Mountains from March 30 to April 5 added to the runoff in the creek.

At 10 and 11 a.m. eastern daylight time on April 5, 1987, the surface weather observed at Schenectady, New York, about 20 miles east southeast of the bridge site, was cloudy with light rain, a temperature of about 48°F, and winds northeast at 12 to 18 knots. Visibility ranged from 7 miles at 9 a.m. to 10 miles at 10 a.m.

<sup>6/</sup> A few urban sections of the Thruway were built with Federal aid highway funds, and tolls are not collected for travel solely in these sections.

The NWS issued at least seven flood statements 7/ and six flood warnings 8/ at the Albany, New York, office from April 4 through April 5, 1987, to advise the public of the rising water and flood conditions along the Schoharie Creek basin. Such statements and warnings issued by the NWS forecast office are disseminated on the National Oceanic and Atmospheric Administration (NOAA) Weather Radio, the NOAA Weather Wire Service, and the National Warning System. 9/ The NOAA weather radio provides continuous 24-hour-a-day weather/river information, which is disseminated in an area over about 40-miles in radius from Albany, New York. The Wire Service disseminates weather warnings, forecasts, and data to the mass news media and other special users for relay to the general public.

### Intensity of Previous Floods

The water level of the Schoharie Creek is recorded and monitored at a stilling well gage 10/ located in Burtonsville, New York, about 13 miles upstream from the bridge. The gaging station, built in 1939, is operated by the U.S. Geological Survey (USGS) and provides data on flows in the lower reach of the Schoharie Creek.

USGS officials stated during the NTSB public hearing in July 1987 that the quality of the Burtonsville gage record is rated as "good throughout," except during periods of ice cover, which reduces the accuracy of the gage.

On April 5, 1987, the water level (gage height) and discharge recorded at the Burtonsville gage peaked at about 8:45 a.m., as indicated below:

Table 1.--Discharge data for Burtsonville gage,  
April 5, 1987

<u>Time (Hours)</u>	<u>Gage Height (Feet)</u>	<u>Discharge (cubic feet/second (cfs))</u>
1:15 a.m.	8.40	39,500
4:15 a.m.	9.22	46,400
7:15 a.m.	10.33	56,400
8:45 a.m.	11.23	64,900
9:00 a.m.	11.07	63,400

7/ Flood statements are warnings for the public of an impending crisis.

8/ Flood warnings are messages to inform people that a flood is imminent or already occurring in an area.

9/ The National Warning System (NAWAS) is an interstate and intrastate telephone hotline operated by the Federal Emergency Management Agency (FEMA), which disseminates information to local municipalities.

10/ A stilling well measures the average static height of a body of water.

After 8:45 a.m., the gage height and discharge rate decreased at Burtonsville, but the flow did not peak at the Schoharie Creek Bridge until some time later.

Using procedures outlined by the Water Resource Council (WRC) in Bulletin No. 17B, the USGS used Burtonsville gage data (1940 to 1987) to determine the flood frequency along the Schoharie Creek. The data indicated that a flow of 64,900 cfs can be expected once every 70 years. (See figures 3 and 4.) Those findings are summarized below in tables 2 and 3. Table 3 ranks all floods in the period of record (1940-1987) greater than 30,000 cfs at the Burtonsville gage.

Table 2.--Flood frequency for Burtonsville Gaging Station 11/  
(1940-1987)

<u>Annual Exceedance Probability</u>		<u>USGS Estimate (cfs)</u>
0.50	(2 years)	20,900
0.20	(5 years)	32,500
0.10	(10 years)	40,700
0.04	(25 years)	51,700
0.02	(50 years)	60,100
0.01	(100 years)	68,900

Table 3.--Floods greater than 30,000 cfs  
during 1940-1987, Burtonsville Gaging Station

<u>Date</u>	<u>Discharge (cfs)</u>
3/31/60	30,100
4/04/60	30,500
4/25/83	31,000
01/09/78	32,800
03/14/77	35,500
03/15/86	37,400
03/31/51	37,900
04/06/84	39,400
10/17/77	39,500
11/09/77	40,400
03/22/80	54,700
04/05/87	64,900
10/16/55	76,500

### Seismic Activity Near Bridge Site

Data obtained from Woodward-Clyde Associates, a consulting firm that operates a network of sensors to detect seismic activity in New York State, indicate that, between April 1 and

11/ Period of record 1940 - 1987. Analysis included floods of 1936 (58,000 cfs) and 1938 (55,000 cfs).

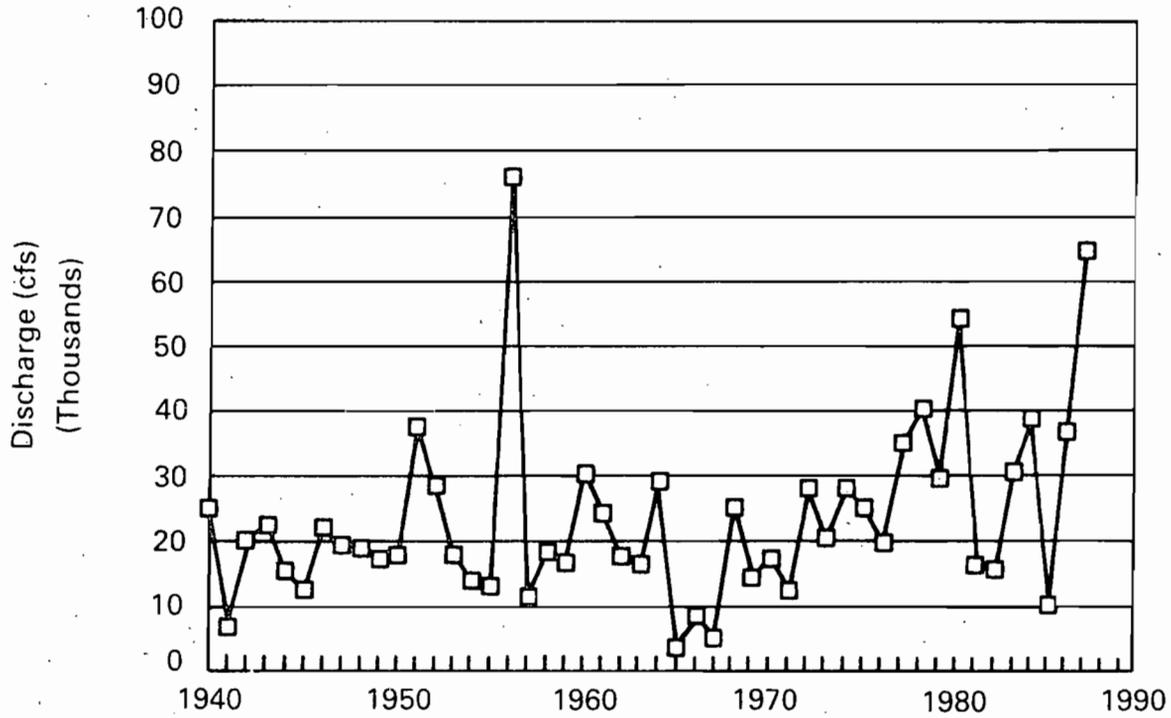


Figure 3.--Annual peak discharges at Burtonsville Gage.

From RCI.

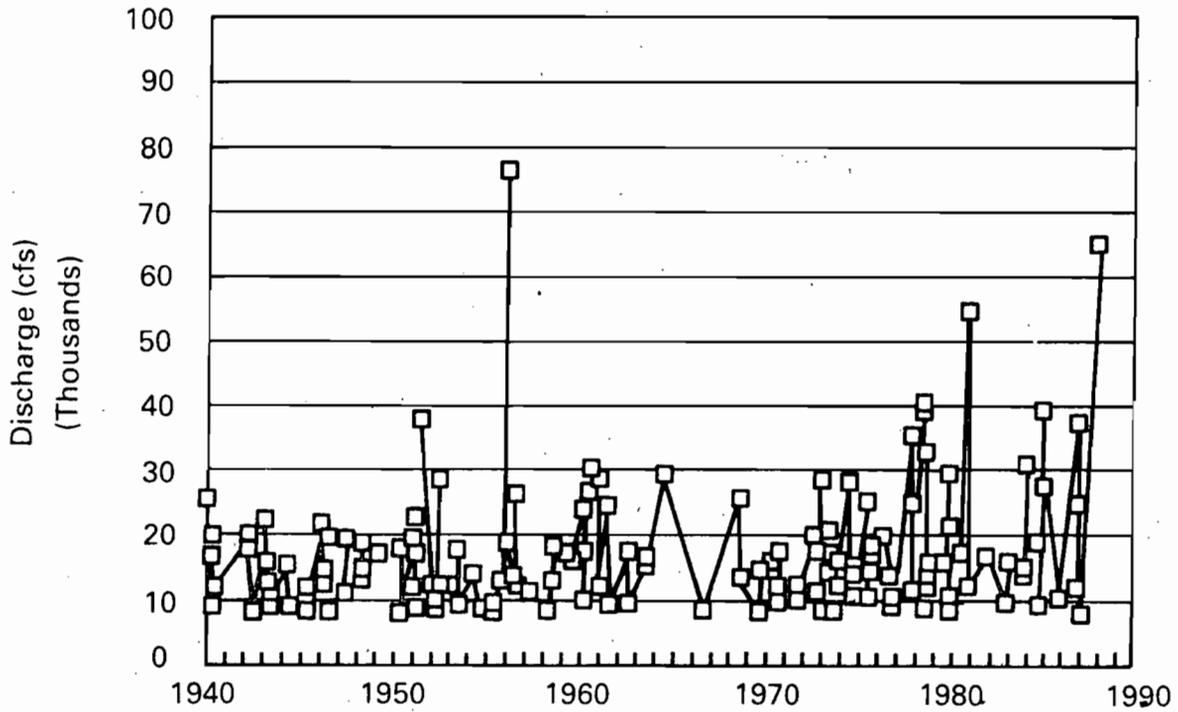


Figure 4.--All floods greater than 8,000 cfs at Burtonsville Gage.

April 6, 1987, no seismic activity was noted at the two sensors in Gloversville and Rotterdam, New York, located about 7 and 10 miles, respectively, from the bridge site. The largest earthquake ever recorded in the vicinity of the bridge site occurred in October 1985, near Amsterdam, New York, and registered 2.7 on the Richter Scale.

### History of Structures Over the Schoharie Creek

Between 1823 and 1917, the Erie Canal operated over the Schoharie Creek, about 4,000 feet downstream of the bridge site. A stone arch aqueduct was built in 1841 to accommodate barges on the canal. Four dams were also built across the Schoharie to provide water for the canal and aqueduct. These dams, one of which was built on piles, 12/ were damaged or destroyed on numerous occasions by floods. However, the aqueduct, which was built on a foundation of spiles 13/ and limestone, withstood damage from flooding for over 100 years until 1940 when the first of its fourteen 40-foot arches collapsed due to undermining.

Between 1880 and 1930, three other bridges were constructed over the Schoharie Creek in the vicinity of the Schoharie Creek Bridge. The State Route (SR) 161 bridge, located at Mill Point, about 6 miles upstream of the Schoharie Creek Bridge, was built in 1927. The SR 161 bridge was built on spread footings with riprap around the footings. Two other bridges, a railroad bridge and the State Route 5S bridge, located about 2,000 feet downstream of the Schoharie Creek Bridge, were built in 1880 and 1930, respectively. The 5S bridge was built on piles. The railroad bridge was initially built on spread footings in 1880; however, in 1905, it was modified by the construction of intermediate piers to carry heavier loads. In 1909, the western intermediate spread footing pier was undermined during a heavy flood, and the spans supported by it collapsed. The center intermediate pier, which was on piles, collapsed after the other spans collapsed and blocked flow. The bridge was subsequently rebuilt with two piers on spread footings, and the easternmost pier, at the outside of the stream bend, on piles.

Each of these bridges was subject to numerous floods including the floods of 1955 and 1987, the two worst floods of record. The two bridges built partially or totally on piles survived these floods, including the April 1987 flood; the SR 161 bridge did not survive the April 1987 flood. It collapsed into the Schoharie Creek about 6 days after the collapse of the Schoharie Creek Bridge. During the April 1987 flood, after water had risen to the bottom of the SR 161 bridge, local officials closed the bridge to traffic.

12/ Safety Board investigators were unable to determine the structure type of the other three dams.

13/ Spiles--wooden posts used as piles.

### Description of Schoharie Creek Channel Watershed Area

Schoharie Creek is the largest tributary of the Mohawk River. Schoharie Creek flows north into the Mohawk at Fort Hunter and drains an area of about 925 square miles. The upper reaches are mountainous while the lower watershed is made up of rugged, rounded hills. "The topography, coupled with a relatively impervious soil, leads to rapid runoff and high flood flows." <sup>14/</sup> The highest point in the basin is at Hunter Mountain, with an elevation of 4,025 feet above sea level. The lowest point is at the confluence <sup>15/</sup> of the Schoharie and Mohawk River, with an elevation of about 280 feet. Schoharie Creek is 83 miles long.

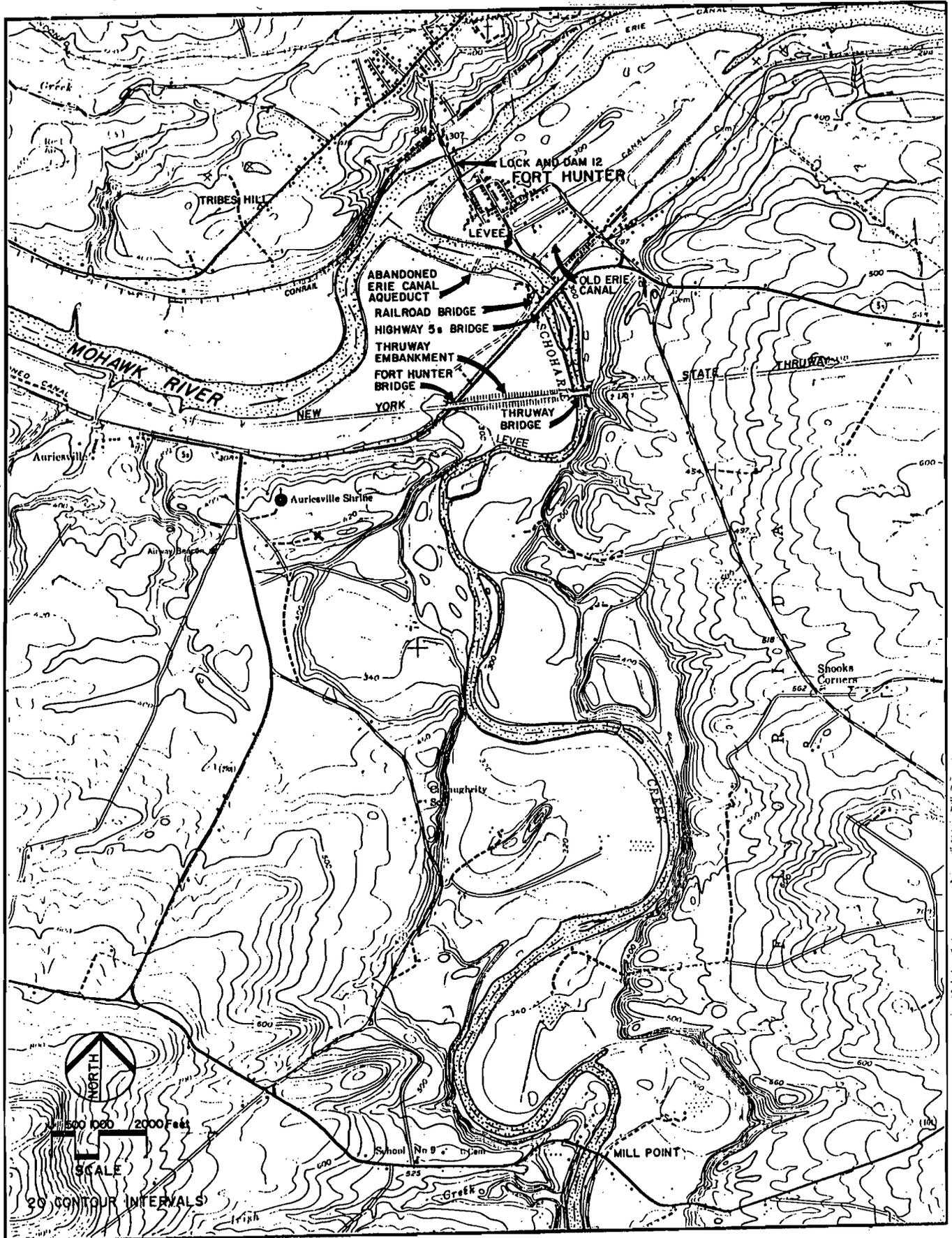
Two dams cross the Schoharie Creek in the upper watershed: the Gilboa Dam (Schoharie Reservoir), operated by the New York City Department of Environmental Protection, and the Blenheim-Gilboa Facility (dam and reservoir), operated by the New York Power Authority (NYPA). (See figures 5 and 6.) Gilboa Dam, located about 58 miles upstream (south) of the bridge site, operates as a water supply reservoir. Gilboa Dam has no facilities for controlling releases to Schoharie Creek and no flood control function. Water in excess of reservoir capacity overflows the face of the dam.

The dam for the Blenheim-Gilboa facility is located about 5 miles downstream of Gilboa Dam. The Blenheim-Gilboa facility is a pumped storage project in which water is pumped to the top reservoir during the hours of off-peak electrical consumption and is allowed to return to the lower level during peak hours through turbines that generate electricity. After the upper and lower reservoirs have been filled to meet the capacity requirements of the project, no additional water is required, except for small amounts to replenish water losses resulting from evaporation and seepage. The Federal Energy Regulatory Commission requires this facility to operate in this manner.

The operation of these dams does influence stream flow during low water periods, if the water reservoir is not full. However, during severe flooding, the effect is minimal. Before the 1987 flood, both dams were near capacity and water equivalent to the inflow was being released in outflow from the dams.

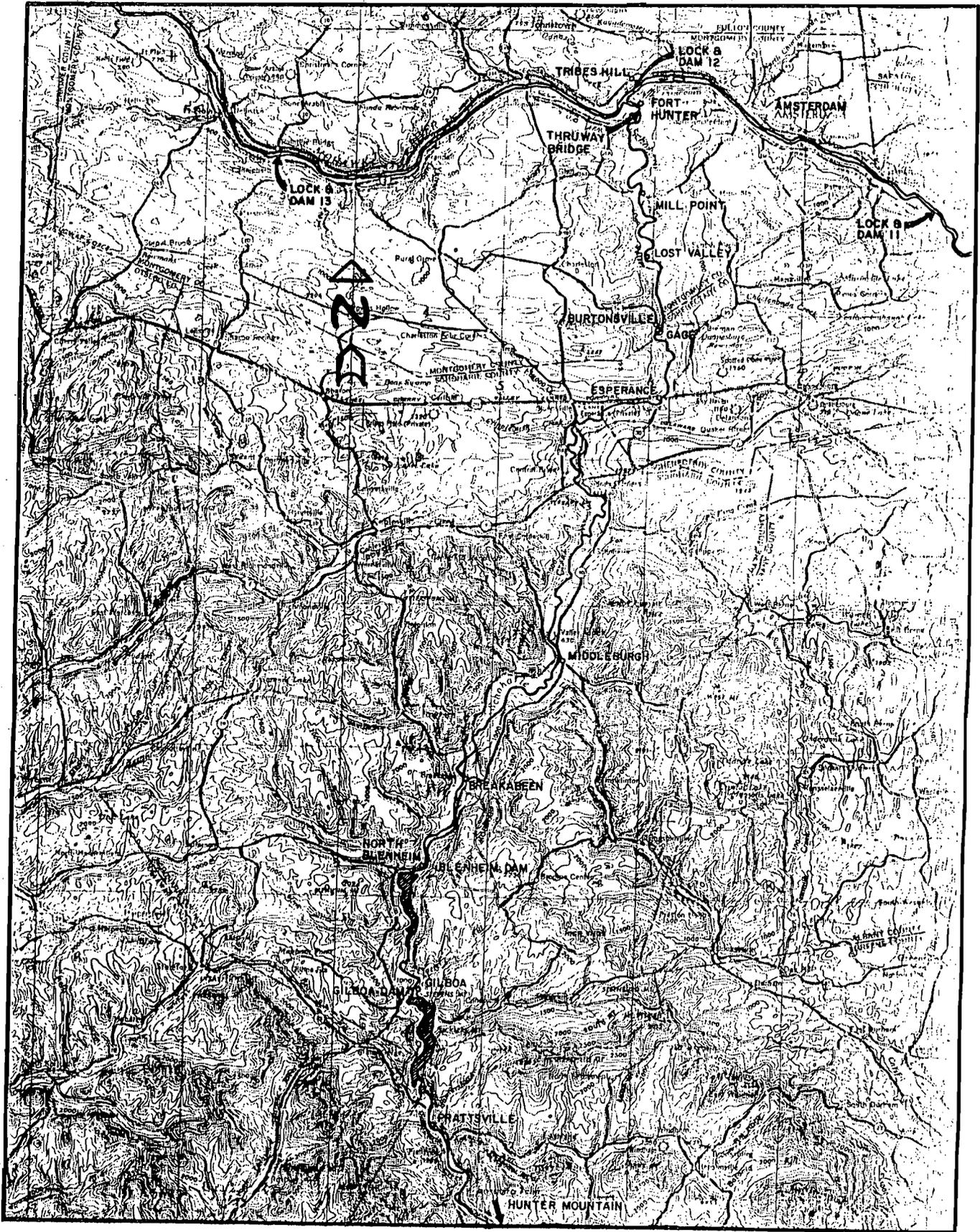
<sup>14/</sup> Cullings, E.S. "Outline of Project for a Comprehensive Study of the Mohawk Watershed," Division of State Planning, Bulletin 22, March 1934.

<sup>15/</sup> Confluence--the juncture of two or more streams.



From RCI

Figure 5.--Schoharie Creek detailed location map from Mill Point to the Mohawk River.



Scale 0 5 10 Miles

From RCI

Figure 6.--Overview and location map, Schoharie Creek drainage basin

Another dam, that at lock 12 of the Barge Canal, 16/ is located immediately downstream of the confluence between Schoharie Creek and the Mohawk River. Because of the dam's location, the elevations of the dam and the two sets of gates that control upstream water elevation can influence the flow in the lower portion of the Schoharie Creek. In 1987, the Barge Canal did not open until April 11. Both gates were raised and the river was unobstructed during the 1987 flood.

### Geology Near the Bridge Site

According to a New York State Geological Survey study, 17/ the present Schoharie Creek bed was created during post-glacial time. At the bridge site, bedrock consists of flat-lying shale and limestone with an average surface elevation of 225 feet above sea level, about 50 feet below the bed of Schoharie Creek. A 40-foot layer of very compact granular glacial till overlies the bedrock. This material is very dense and thick, due to compaction by heavy overlaying glaciers. In addition, large boulders scattered throughout the material hinder excavation and the driving of piles. A thick layer of alluvial material composed of brown sand and a layer of well rounded cobbles 18/ forms the river bed. The materials in the streambed range from gravel-sized pieces to boulders that may be several feet in diameter and weigh 300 to 600 pounds. The sand and cobbles are not permanently fixed to the river bottom, but gradually migrate downstream.

### Bridge Information

The Schoharie Creek Bridge consisted of five spans with nominal lengths of 100, 110, 120, 110, and 100 feet, for a total length between abutments of 540 feet. A schematic plan, developed by Wiss, Janney, Elstner Associates, Inc. (WJE), 19/ from the original drawings is shown in figure 7. The profile drawing from the original design plans is shown in figure 8. Figure 9 shows the pier details based on a WJE drawing.

16/ The Barge Canal, a series of navigation locks and dams on the Mohawk River, is operated by the New York State Department of Transportation (NSYDOT).

17/ Dineen, Robert J. "Schoharie Creek Flood of April 5, 1987, A Preliminary Report," New York State Geological Survey, 1987.

18/ Cobble--A rock fragment between 2.5 and 10 inches in diameter, thus larger than a pebble and smaller than a boulder, rounded or otherwise abraded in the course of transport by water, wind, or ice.

19/ Report on the Collapse of the Thruway Bridge at Schoharie Creek prepared for the New York State Thruway Authority by Wiss, Janney, Elstner Associates, Inc., and Mueser Rutledge Consulting Engineers.

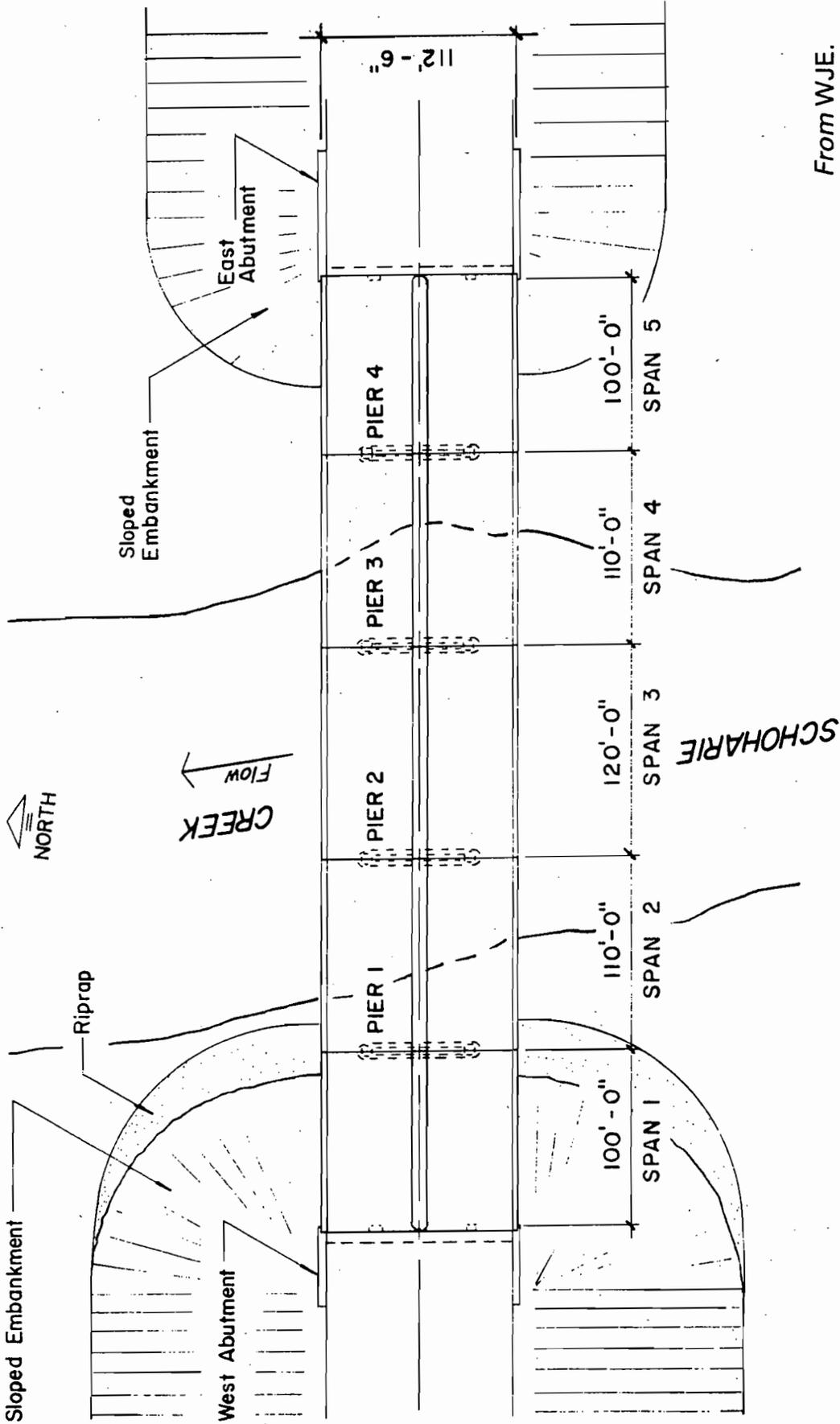
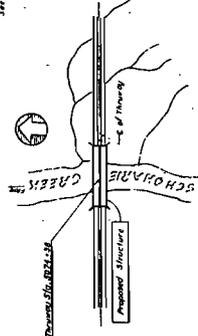
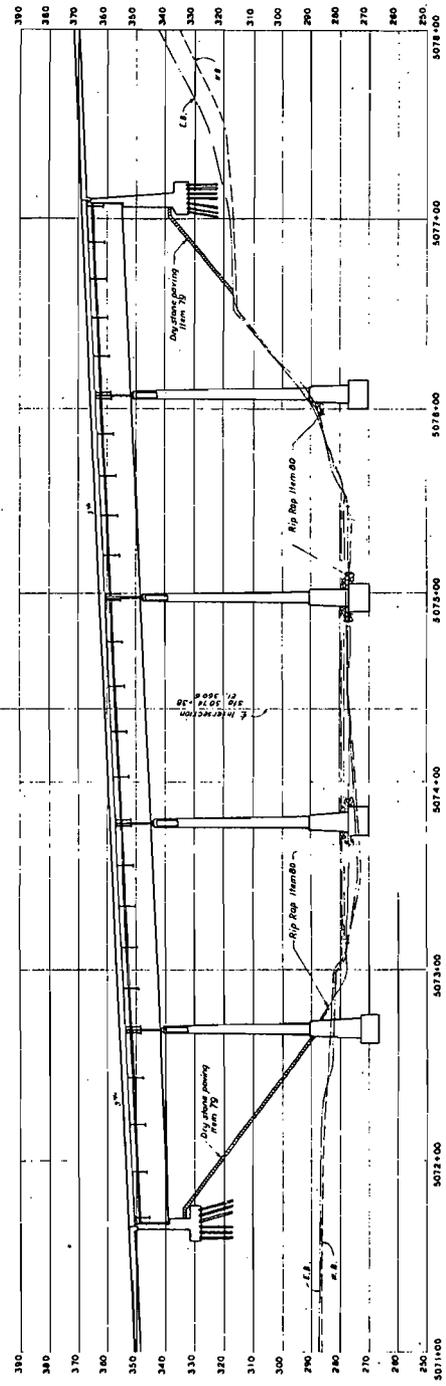
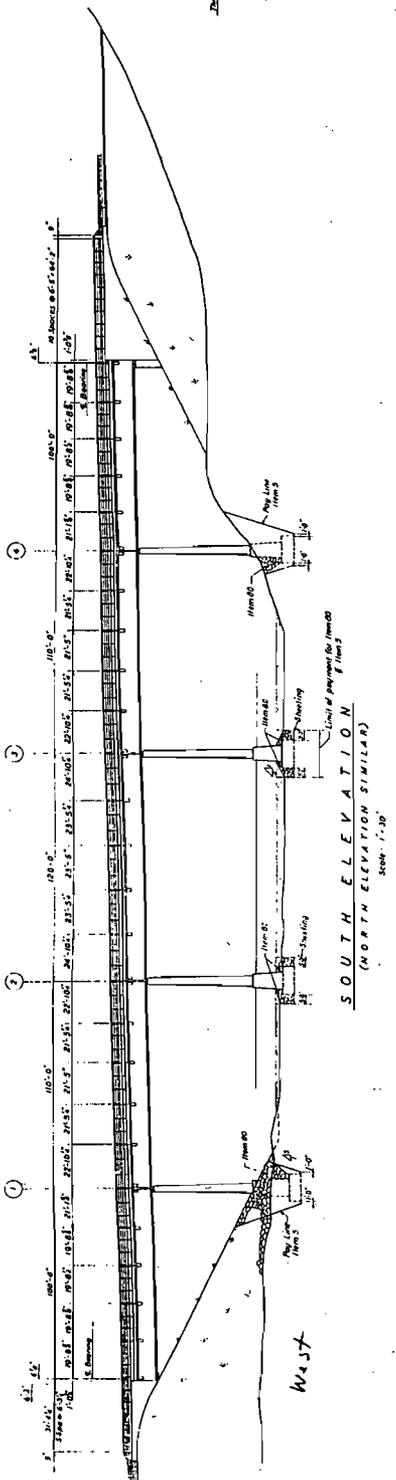


Figure 7.--Schematic plan of bridge.

COUNTY MONTGOMERY  
 SHEET NO. 60  
 N. Y. STATE THRUWAY - MOHAWK SECT. SUB-DIV. 12  
 RANDALL TO FORT HUNTER-SCHOMARE CREEK BRIDGE



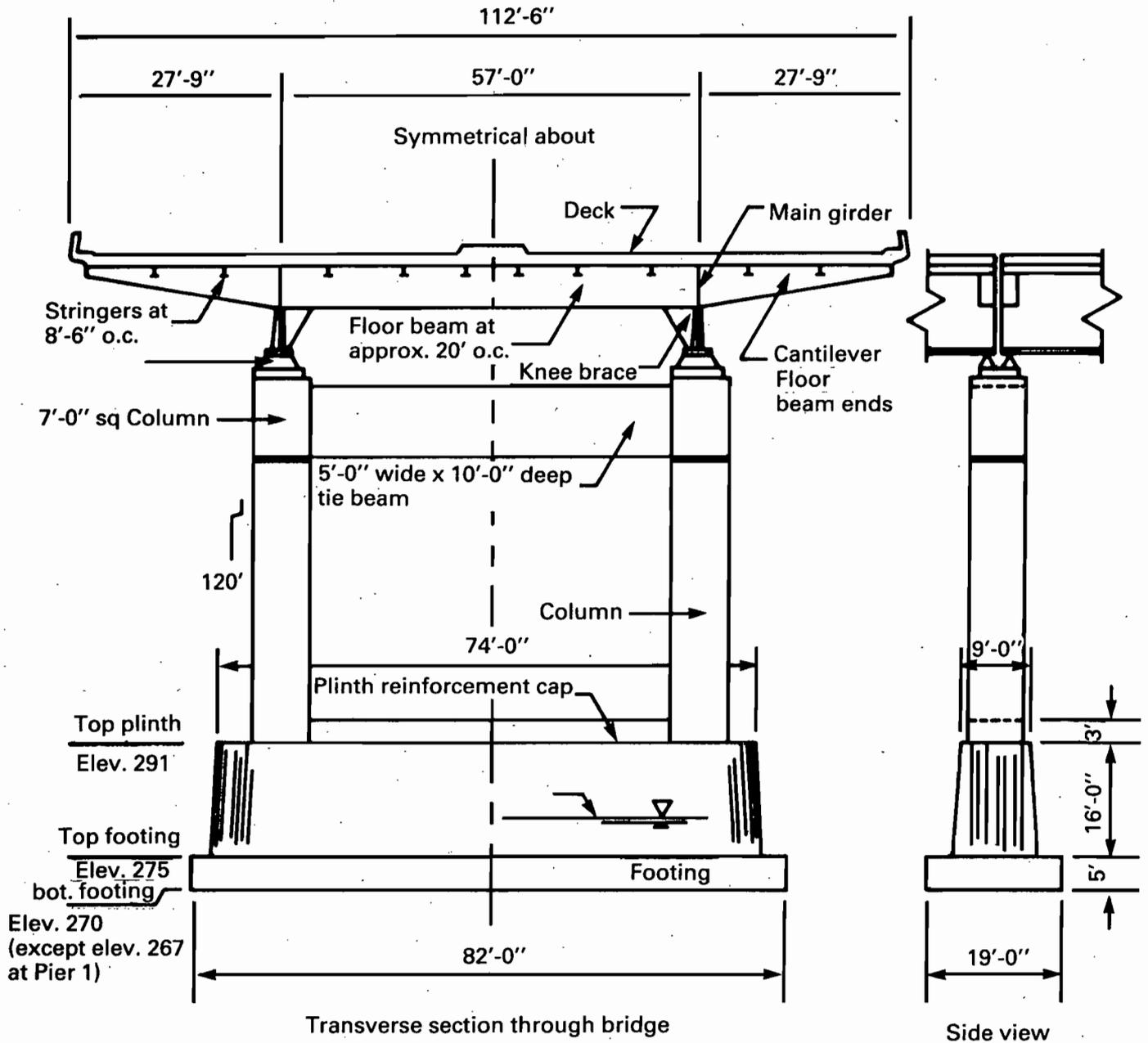
GENERAL NOTES:  
 1. The existing bridge is a 2-span, 140'-0" long (measured from centerline to centerline) structure with a minimum of 12' clearance over the waterway.  
 2. The proposed structure is a 2-span, 140'-0" long (measured from centerline to centerline) structure with a minimum of 12' clearance over the waterway.  
 3. The proposed structure is to be constructed of reinforced concrete with steel reinforcement.  
 4. The proposed structure is to be constructed with a design life of 50 years.  
 5. The proposed structure is to be constructed with a design speed of 45 mph.  
 6. The proposed structure is to be constructed with a design wind speed of 70 mph.  
 7. The proposed structure is to be constructed with a design flood of 100 year return period.  
 8. The proposed structure is to be constructed with a design seismicity of 0.15g.  
 9. The proposed structure is to be constructed with a design temperature of 70°F.  
 10. For structural notes - see sheet 114 & 115



GENERAL PLAN  
 ELEVATION & PROFILE

FIGURE 8. - - ORIGINAL DESIGN PLAN.

DESIGNED BY: R.L. PAUL  
 CONSULTING ENGINEER  
 LICENSE NO. 10000  
 REGISTERED PROFESSIONAL ENGINEER  
 DATE: 10/15/52  
 PROJECT: RANDALL TO FORT HUNTER-SCHOMARE CREEK BRIDGE  
 SHEET NO. 60  
 COUNTY: MONTGOMERY  
 STATE: N.Y.



From WJE.

Figure 9.--Sections of the bridge pier.

The bridge was designed to carry three lanes of traffic each way over a total roadway width, including curbs, of 112 feet 6 inches. However, the bridge carried only two lanes of traffic each way. The bridge was built on an upward, 3 percent grade from west to east, and had an average height of about 80 feet above the creek. Alignment of the bridge deck was straight, and the roadway had no cross slope 20/ except for crowns 21/ on the north and south sides of the roadway for drainage.

Each span of the bridge had a reinforced concrete deck. Spans were reinforced with steel bars for tension. The deck was not connected to its underlying steel supporting members, which were supported by steel bearings on reinforced concrete piers and end abutments. A typical transverse section of the bridge is shown in figure 9. After the bridge was opened to traffic, all four plinths 22/ cracked vertically through the middle. In 1957, the cracked segments of the plinths were reconnected and reinforced with a plinth reinforcement cap.

### Bridge Damage

The collapsed bridge was almost a total loss. (See figures 10 and 11.) The estimated total cost for the construction of the replacement bridge, detours, forensics, overtime, and lost Thruway revenues was about \$44 million dollars.

On April 7, 1987, NYSTA contracted with WJE with a subcontract to Mueser Rutledge Consulting Engineers (MRCE) to investigate the collapse of the Schoharie Creek Bridge. WJE planned and monitored the removal and demolition of the structural components and MRCE studied the soils and foundations for the bridge.

Two weeks after the collapse, WJE had a diver explore the area south of the bridge piers. Within 3 weeks of the collapse, WJE inspected the wreckage and had aerial photographs of the bridge site taken. Before removing the bridge materials, WJE studied the wreckage distribution and developed a hypothesis of the collapse sequence. WJE then had a cofferdam built around the eastern two-thirds of the bridge debris to drain the area and expedite the documentation and removal of the bridge debris.

20/ Cross slopes--A curve, plane, or combination of slopes built across a highway to drain water from a high point on the road to the side of the road or into a drainage system.

21/ Crown--An element of the highway cross section created by the raising of the centerline of the roadway above its edges.

22/ Plinth--The enlarged, or spread-out, lower portion of the pier, which distributes the load to the spread footing.

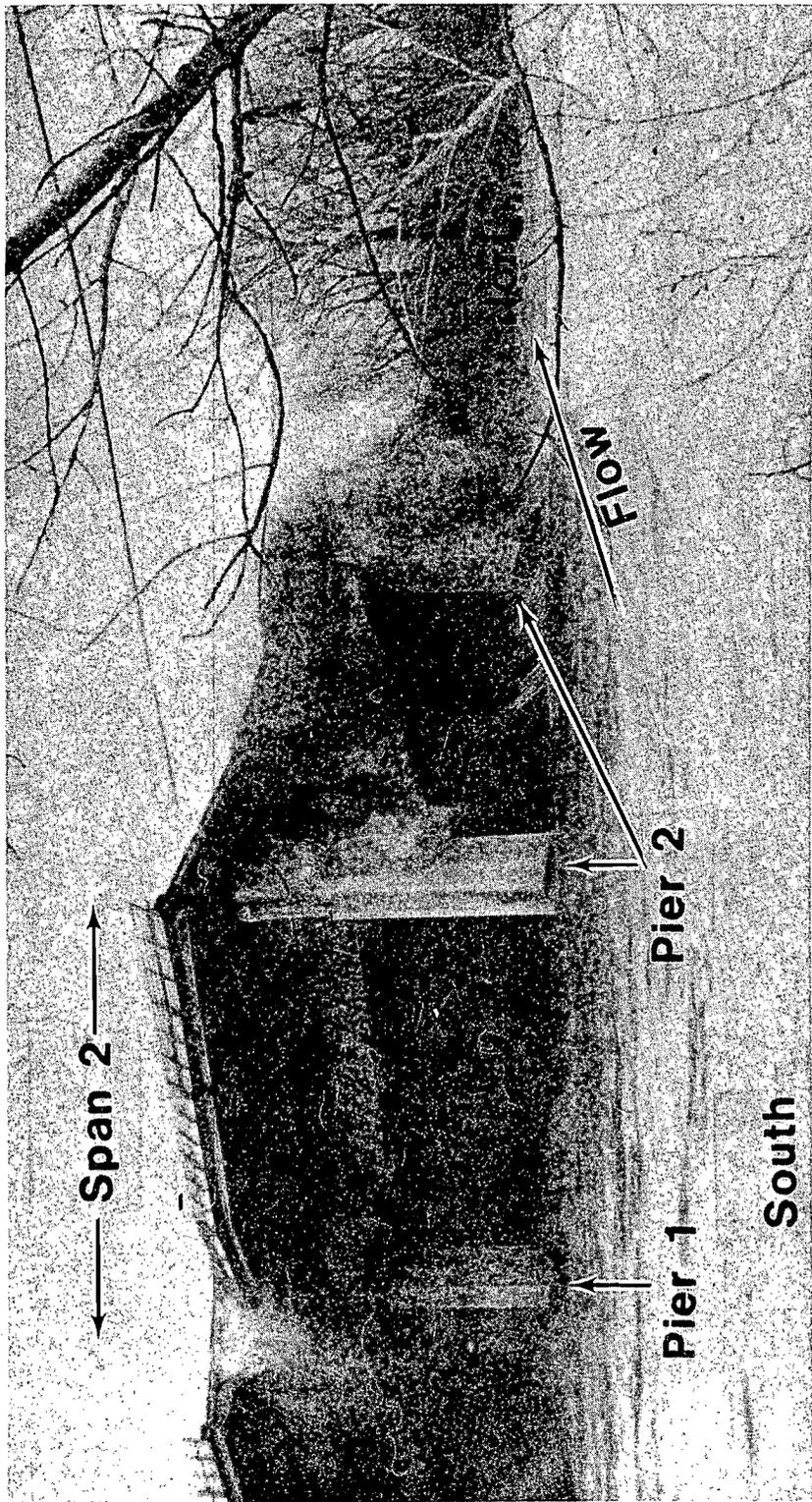
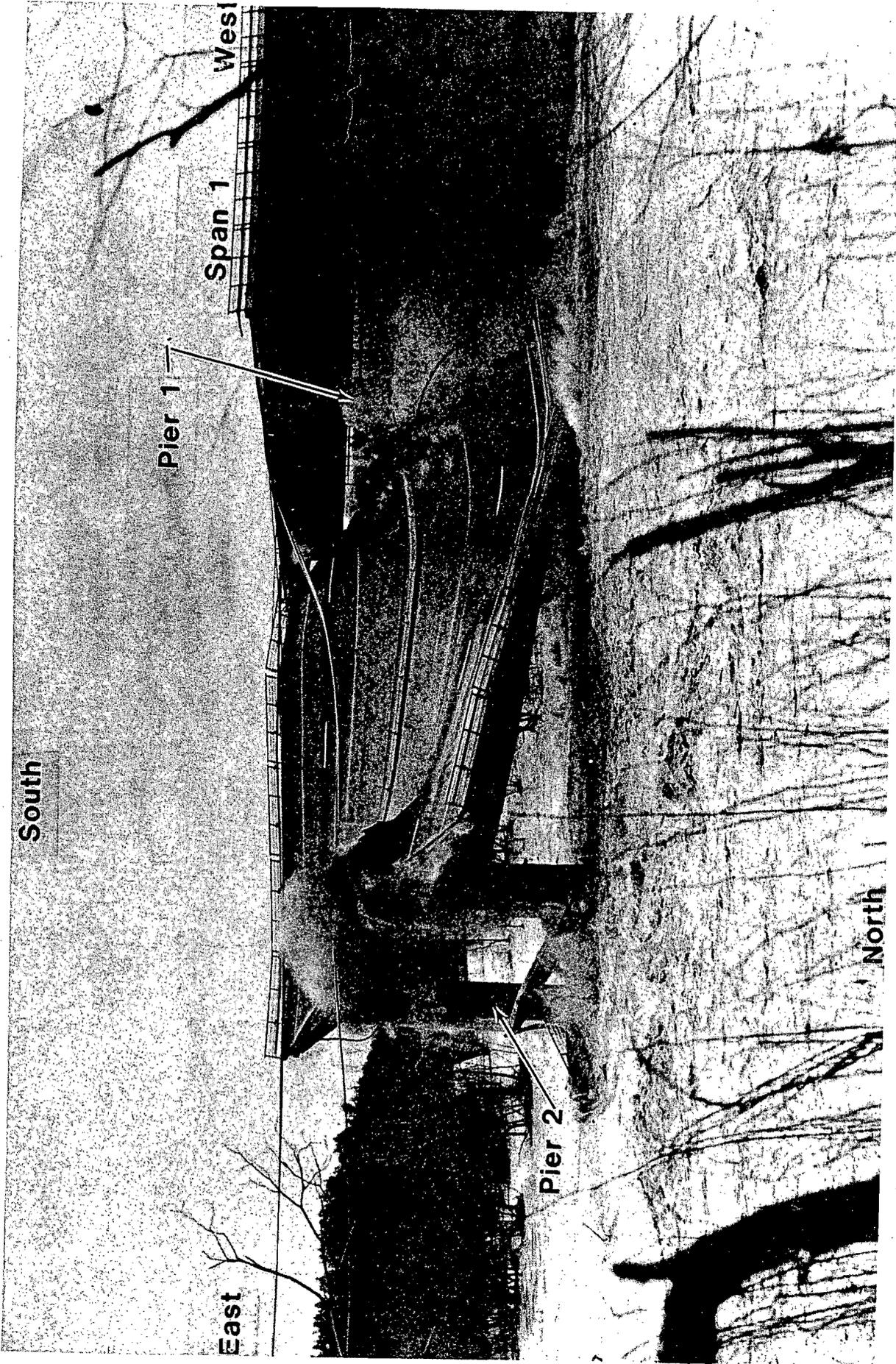


Figure 10.--Photograph looking northwest during the collapse of span 2.



Courtesy of Sid Brown Schenectady Gazette.

Figure 11.--Collapse of span 2.

After the collapse, span 1 was dropped using light charges of dynamite placed at the south column of pier 1. Span 5 was tied back to the east abutment and dismantled. The west abutment was removed and about 1/3 of the backwall, the piles, and most of the wing walls on the east side were salvaged and incorporated into the replacement bridge.

### Wreckage Documentation-Superstructure Components

WJE used field survey measurements and photographs of the collapsed debris to develop a detailed site plan. The positions of members of spans 2, 3, and 4 in the creek are shown in figures 12 and 13.

WJE reported that the ends of several of the main girders 23/ were bent along a line extending from the bottom of the floor beam 24/ to the junction between the bottom flange angles 25/ and the first interior stiffeners. 26/ Bends near the ends of the first cover plate 27/, along a line normal to the axis of the girders, were also observed on many of the girders. The west ends of both the north and south girders of span 4 had major bends at these locations. Full or partial fractures of the girders around the rivet holes were observed in both the top and bottom of the girders. (See figure 14.)

WJE also found that the fractures along the rivet holes had separated some of the connections between the floor beam girders and stringers of the floor beams. The north girder of span 3 was bent at the bottom flange of the floor beams over a substantial part of its length. The diaphragms 28/ between these floor beams and the bottom flange of this girder were distorted and the connections were fractured.

23/ Main girder--a flexural member that is the primary support for the structure, and that often receives loads from floor beams and stringers.

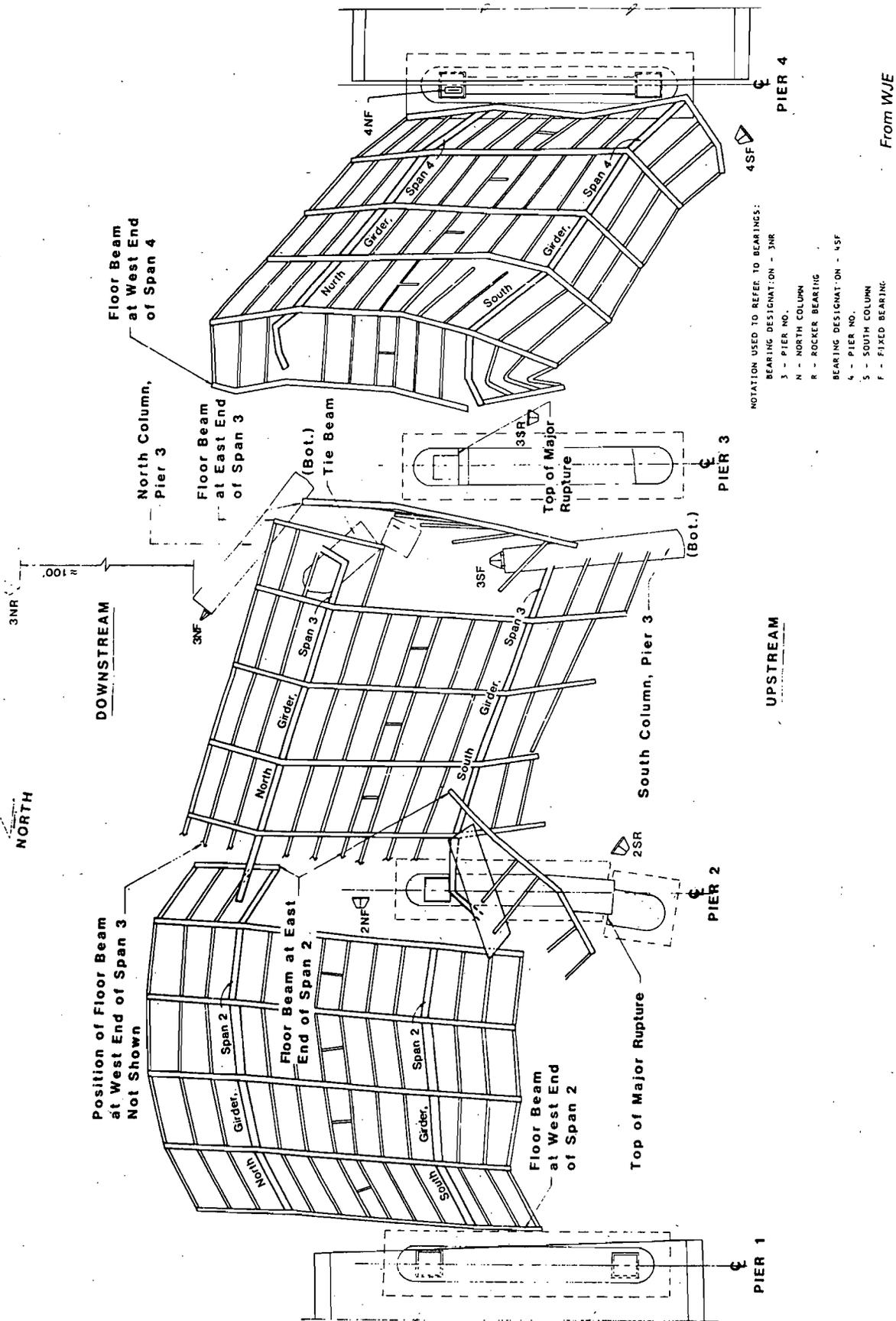
24/ Floor beam--a beam or girder located transversely to the general alignment of the bridge with its ends framed upon the columns of bents and towers or upon the trusses or girders of superstructure spans. A floor beam at the extreme end of a girder or truss span is commonly termed an end floor beam.

25/ Flange angle--an angle used to form a flange element of a built-up girder, column, strut, or similar member.

26/ Stiffener--an angle, tee, plate, or other section riveted, bolted, or welded upon the web of a plate girder or other "built-up" member to transfer stress and to prevent buckling or other deformation.

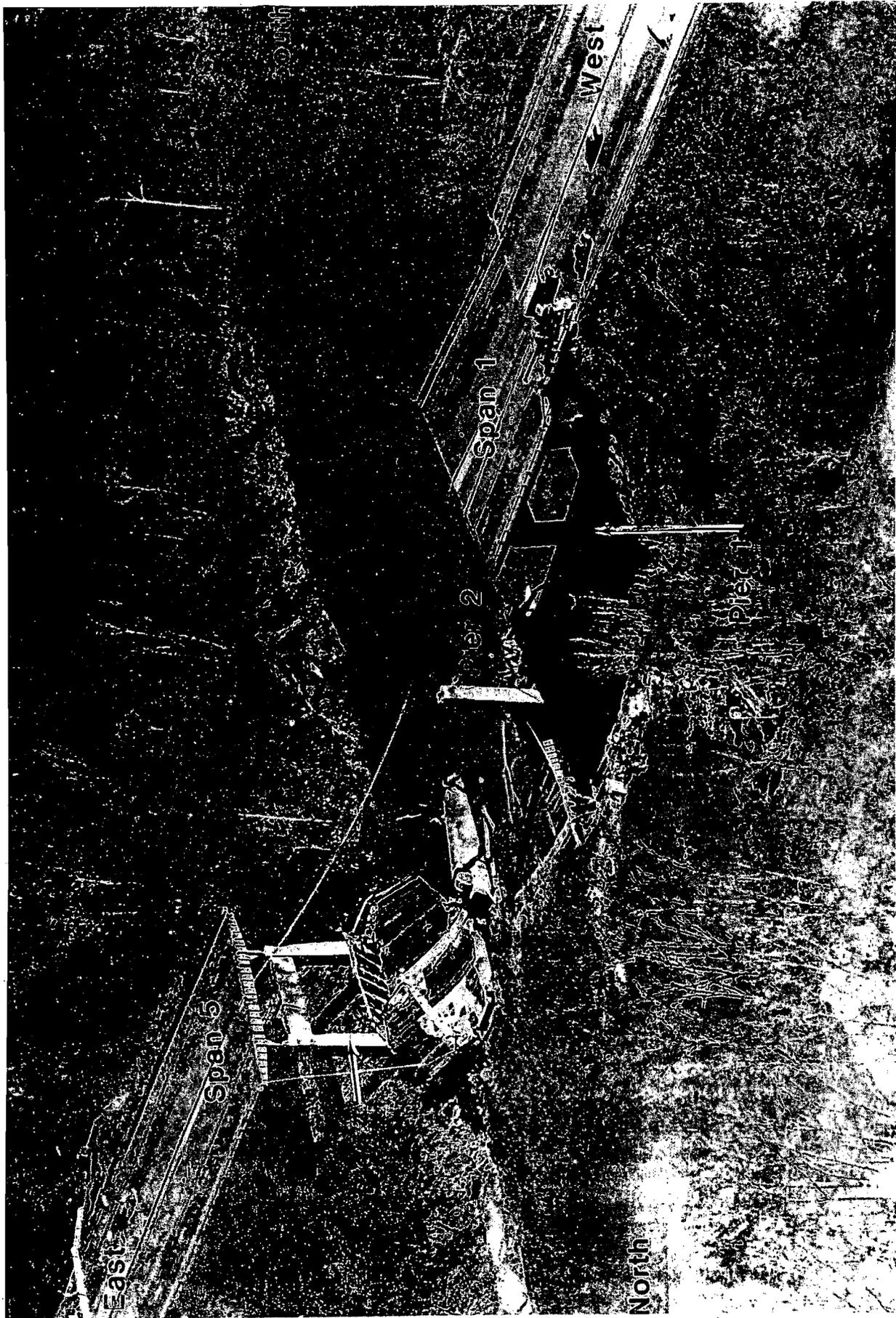
27/ Cover Plate--a plate used with flange angles or other structural shapes to provide additional flange section upon a girder, column, strut, or similar member.

28/ Diaphragm--a reinforcing plate or member placed within a member or deck system, respectively, to distribute stresses and improve strength and rigidity.



From WJE

Figure 12.--Plan view of collapsed spans 2, 3, 4.



Courtesy of Sid Brown Schenectady Gazette.

Figure 13. -- Wreckage documentation.



Figure 14.--End of the south girder of span 3 near pier 2.

Bearings.--At the end of both main girders of each span of the bridge there was a fixed bearing on the east side and a rocker bearing on the west side. <sup>29/</sup> (See figures 15 and 16.) With 2 main girders for each of the 5 spans, 20 bearings supported the bridge.

WJE engineers recovered and examined 16 of the 20 bearings, including all 4 of the bearings from pier 3. The fixed bearings on pier 3 remained attached to the tops of the columns, but the rocker bearings did not.

When WJE examined the four bearings of pier 3, they found that the most notable surface feature was the similar rounding of the south ends of all of the bearings. The downward deformation at the ends was about 1 to 2 inches and extended over lengths of 3 to 5 inches. The fixed bearings were more deformed than the rocker bearings. WJE stated that the rounding was probably caused during the collapse by the sliding and rotation of the ends of the girders supported on pier 3.

WJE found that the fixed bearing on pier 4 that supported the north girder of span 4 was also rounded at its south end. Based on the marks on the girders of spans 4 and 5, WJE stated that the north girder of span 4 had undergone substantial rotation before it came off the bearing.

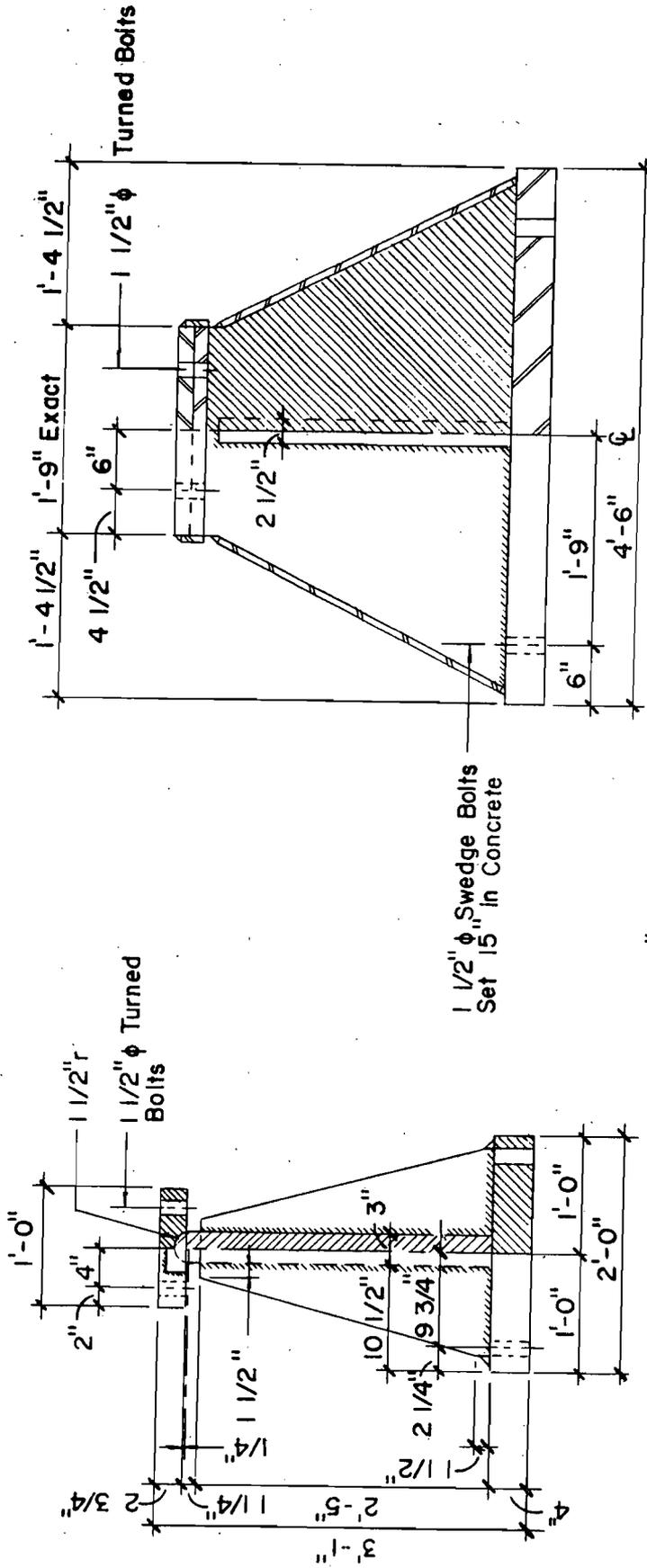
#### Wreckage Documentation-Substructure Components

Pier 3.--A diver performed a cursory examination of pier 3, the first pier to collapse, several days after the bridge collapsed. However, a more extensive examination was performed by WJE, MRCE, NTSB, and others after the site was dewatered.

Pier 3 settled 3.4 feet at the centerline of the south column, but its north end was at its originally constructed elevation. Both columns of pier 3 fell to the west side of the pier.

The plinth of pier 3 had broken into two pieces. WJE reported that the break began near the junction of the plinth reinforcement and the north column and extended down, near the center of the plinth. The break intersected another vertical crack that extended through the footing. WJE stated that the lower crack had occurred sometime before the bridge collapsed, as indicated by the dirty interface between the crack.

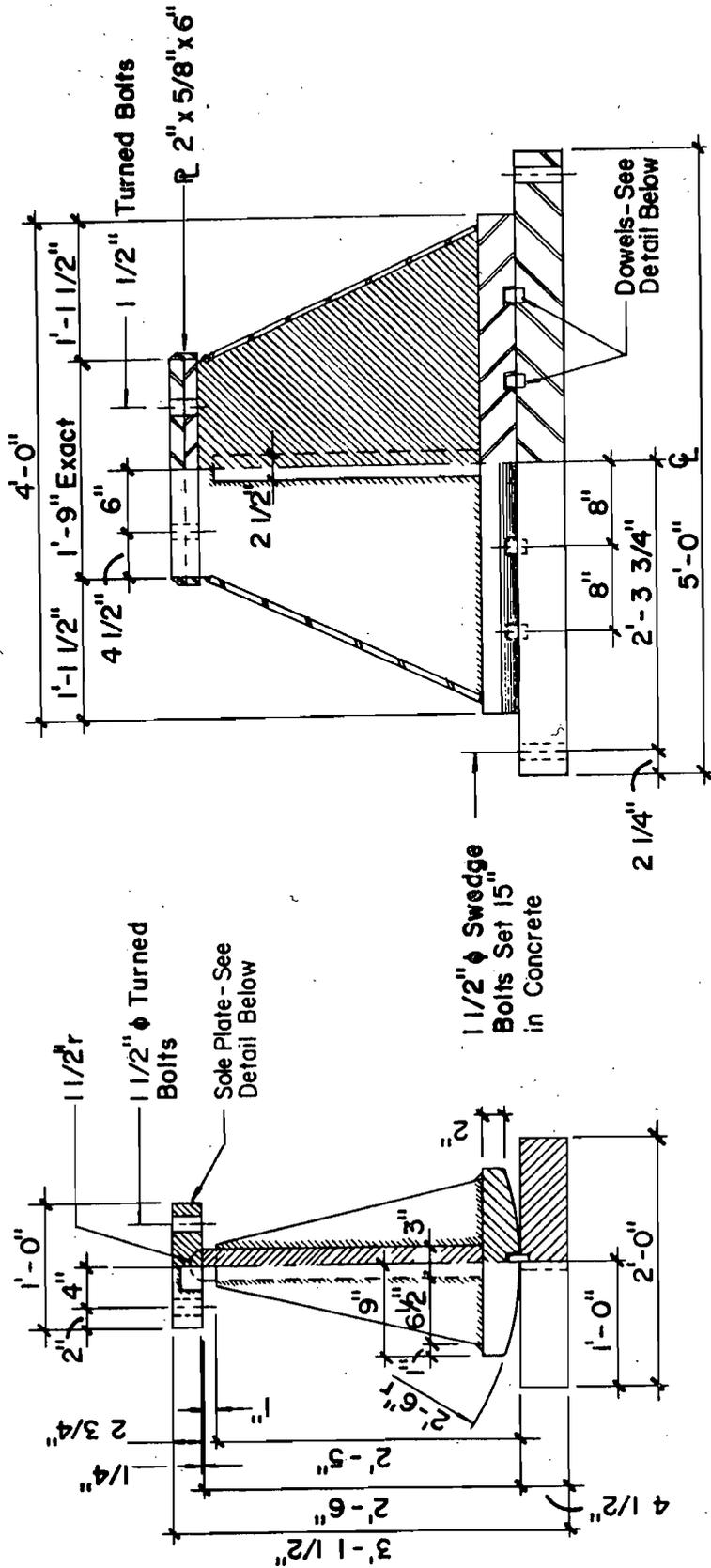
29/ Rocker bearing--a movable support with convex contact surfaces attached to the bridge's main supporting members and the substructure at the expansion ends of a bridge span to provide for longitudinal movement resulting from temperature changes, creep, shrinkage, and live loads.



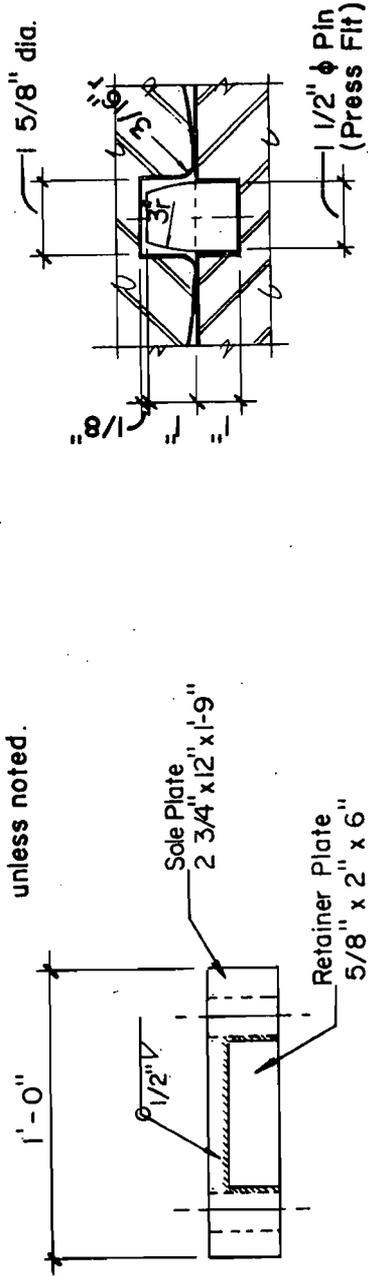
Note: All welds 1" fillet welds unless noted.

From WJE.

Figure 15.--Fixed bearing details.



Note: All welds 1" fillet welds unless noted.



From WJE.

Figure 16.--Expansion bearing details.

The elevation of the interface between the base of the north column and the top of the plinth was 290.9 feet, about the same as its original elevation in 1955. The south column base was estimated to have been at elevation 287.6, about 3.4 feet lower than its original construction elevation.

The soil beneath the extreme upstream end of the footing had been eroded. The upstream end of the footing had dropped into a scour hole that was 9 feet deep. (The deepest part of the hole was located about 3 feet west of the upstream end of the footing.) The downstream end of the pier had not moved. (See figure 17.)

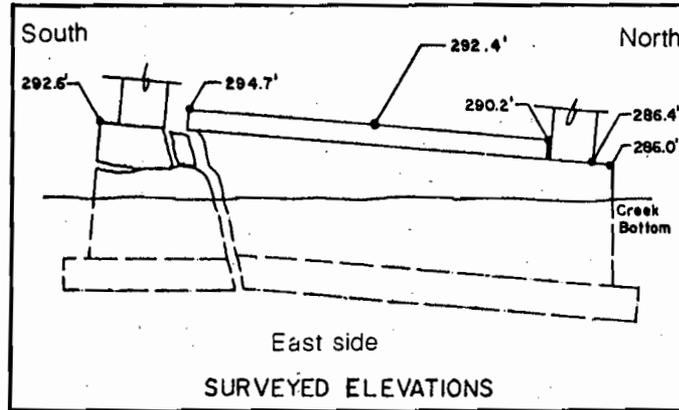
Pier 2.--Pier 2 collapsed about 90 minutes after pier 3. The video film of the collapse of pier 2 and span 2 showed that before the collapse, the debris from spans 3 and 4, which partially blocked the channel, diverted much of the flood water toward pier 2.

WJE found extensive erosion of the soil under pier 2. Survey measurements indicated that the pier had settled along its entire length and was also tilted toward the west. The north end had settled about 5 feet below its originally constructed elevation. The north column, as measured at the interface between the column base and the top of the plinth, settled an average of 3.9 feet. The south column settled an average of 2.1 feet. The north column remained standing after the collapse, and was removed during the demolition work.

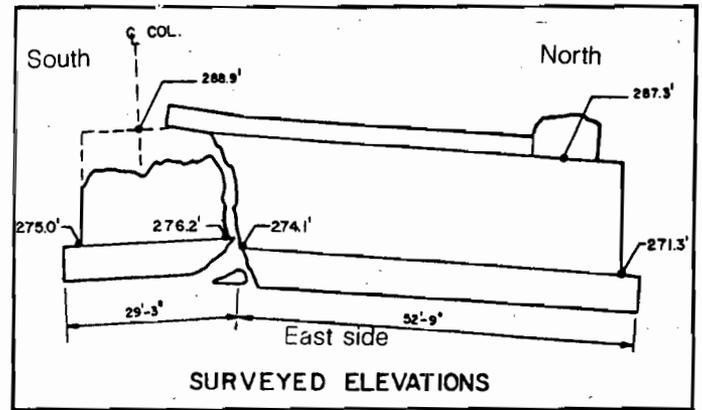
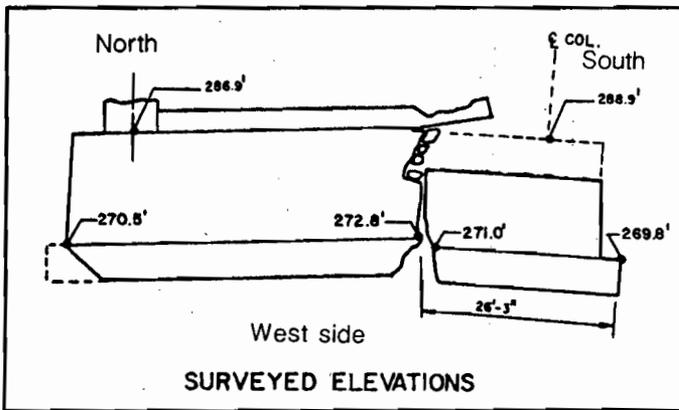
The plinth of pier 2 had broken into two pieces. WJE reported that the break began about 6 feet north of the junction of the plinth reinforcement and the south column, extending downward vertically through the plinth and footing. Measurements of the top of the eastside footing adjacent to the break showed that this area was at its original as-constructed elevation of 275.0 feet. The south column separated from the plinth at the bottom of the column steel reinforcement dowels, which were embedded 5.5 feet below the top of the plinth.

In addition, pier 2 rotated to the west about the longitudinal axis of the plinth. The elevation of the western end of the south pier segment was 5 feet lower than the eastern end, while the north pier segment had a change of 1.2 feet in height along its 19-foot width. The northwest corner of the footing was broken off at about the edge of the plinth.

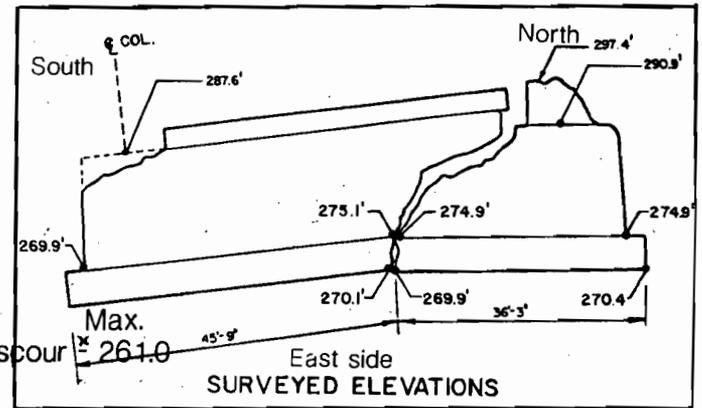
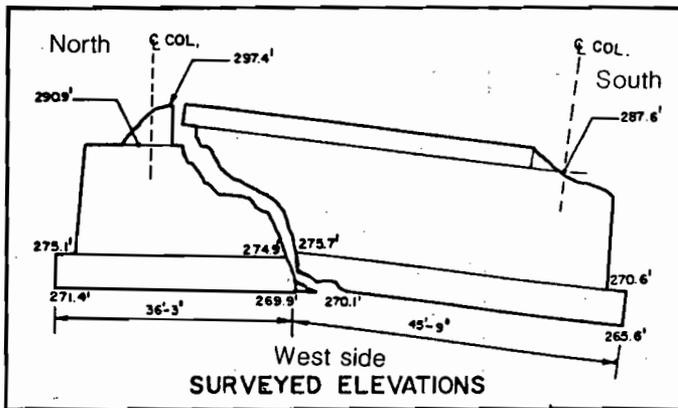
Pier 1.--Pier 1 tilted significantly to the north and east. WJE reported that the columns were out-of-plumb. In addition, the plinth was broken into two pieces, with the break beginning at the junction of the plinth reinforcement and the south column, and extending downward into the plinth. The break was wide at the top and WJE projected that it extended down through the footing. A horizontal separation in the plinth about 6 feet below the base of the south column appeared to have begun along a construction joint. The north end of the footing at pier 1



Pier 1



Pier 2



Pier 3

Based on WJE drawings.

Figure 17.--Elevations of piers 1, 2, and 3 after the collapse.

settled more than 5 feet. The north column of pier 1 had settled about 4 feet. WJE indicated that the south end of pier 1, below the horizontal separation, had not moved even though debris from the collapse of pier 2 had directed more water towards pier 1.

Pier 4.--The plinth and footing of pier 4 did not exhibit any damage. Survey measurements indicated that pier 4 had not moved. The elevation of the base of the north and south columns had not changed appreciably from the time of construction.

### Design of Bridge

Planning for Highway/Bridge.--The initial planning for the highway that later became the Thruway was started in the early 1940s by the New York State Department of Public Works (DPW), predecessor to the New York State Department of Transportation (NYSDOT). The two DPW offices involved in the bridge design process were the Bridges-Grade Crossings-Structures Division at the Albany Headquarters (DPW-HQ) and the District Engineer's office in Utica (DPW-DE). In 1948, DPW began selecting routes and conducting preliminary geological and soils studies for a 14.43-mile segment of the Thruway that would include the Schoharie Creek Bridge. In March 1951, DPW-HQ contracted with Madigan-Hyland (M-H), an engineering consultant company, to develop preliminary plans, detailed design plans, specifications, quantity and cost estimates, and right-of-way needs for that segment of the Thruway. The work was to be completed in 14 months, by May 1952.

During the time in which the Thruway was being built, the Federal Interstate Highway Program <sup>30/</sup> was being formulated. At that time, it was unclear if Federal-aid funds would be available to support the construction of the Thruway. However, all of M-H's plans and estimates for the highway and bridge were to be subject to the approval of the U.S. Bureau of Public Roads (BPR) (now the Federal Highway Administration). Because the Thruway was constructed primarily with private funds, the BPR was not required to approve the plans and estimates; however, it did provide a partial review and offered comments regarding this bridge. DPW was involved in the oversight of M-H throughout the preliminary planning.

The detailed designs of the Schoharie Creek Bridge and several other bridges in the M-H contract were subcontracted to E. Lionel Pavlo, Consulting Engineer (Pavlo). Pavlo accomplished the work during the summer and fall of 1952. A former M-H official who worked on this project recalled that Pavlo was paid both to design and to check the design calculations. He also indicated that when the plans were passed through M-H and

<sup>30/</sup> Federal Interstate Highway Program - The Federal program established in 1956 to build the Interstate Highway System, which typically funded roads with 90 percent Federal funding.

DPW for review, "the review was more or less cursory and I suppose there was more attention paid to the engineer's quantities and estimates than to the details being shown on the contract plans."

Preliminary Design of Bridge--Preliminary design of the bridge included selecting the location, sizing 31/ the bridge, and deciding on structure type. In regard to these tasks, the DPW contract with M-H stated that:

- o If piles were required, the type and load per pile was to be indicated on the plans.
- o The structures were to be designed in accordance with the 1949 Standard Specifications for Highway Bridges of the American Association of State Highway Officials (AASHO--now AASHTO, the American Association of State Highway and Transportation Officials). 32/

Section 3.1.1 of the AASHO Specifications stated in part:

For the determination of the waterway area to be provided by any drainage structure, a careful study shall be made of local conditions, including flood height, flow and frequency, size and performance of other openings in the vicinity carrying the same stream, characteristics of the channel and of the watershed area, climatic conditions, available rainfall records and any other information pertinent to the problem and likely to affect the safety or economy of the structure.

Section 3.5.1 of the AASHO Specifications stated in part:

At locations where unusual erosion may occur and the soil conditions permit the driving of piles, they, preferably, shall be used as a protection against scour, 33/ even though the safe bearing resistance of the natural soil is sufficient to support the structure without piling.

In May 1951, M-H provided a location plan that called for one soil boring 34/ at each bridge pier. DPW made these borings in July and August 1951. At the planned location of pier 3,

31/ Sizing refers to determining length, number of spans, and width, among other factors.

32/ The 1949 AASHO Specifications for Highway Bridges were to be used by State highway authorities in design specifications for bridges, and as a reference for bridge engineers. Excerpts from relevant sections of the 1949 AASHO Specifications appear in appendix D.

33/ Scour--the downward erosion of a streambed by the stream, especially during floods.

34/ Borings--a technique used to obtain samples of materials below the surface such as standard penetration resistance measurements, rock drilling of cores, wash borings, or chopping, to determine soil composition.

the boring was made at the north end and started at an elevation of 276.9 feet. The top 3 feet of the boring showed coarse gravel and boulders. The next 3 to 9 feet were composed of silt with some sand and a trace of gravel. From 9 to 14 feet, there were compact silt and sand with some gravel. The next 20 feet of borings contained sand with some silt and gravel. Similar material was found at the other piers. (See figure 18.)

According to M-H correspondence, the preliminary plans for the Schoharie Creek Bridge included a fact sheet on the hydraulics (hydraulic sheet), the preliminary drawing, and the preliminary estimate. The preliminary drawing showed the type of construction planned, the number of spans, bridge cross sections, type of foundations, elevation of footings, allowable soil pressures, and other basic design requirements, such as heavy reinforcement of the plinth.

The hydraulic sheet, which was developed by the M-H highway design division, indicated that erosion could be expected on the banks of the Schoharie Creek and in its streambed. For a 575-foot-long structure with five spans of 115 feet each, "the extreme high water elevation" was 290 feet with velocities of 10 to 12 feet per second (fps). The hydraulic sheet indicated that the stream reached high water elevation rapidly and then receded rapidly. However, no design discharge was indicated. Piles were recommended "under abutments only".

No piles were shown on the design plans under the footing for the bridge piers. DPW and M-H, based on initial soil boring data, felt that the bearing capacity of the glacial till would be sufficient to support the bridge structure. A former M-H employee recalled that piles "were never considered for the pier foundations...based on the material that showed up on the borings...."

In November 1951, M-H submitted two versions of the preliminary plans for the bridge design to DPW-HQ for its review. One plan called for a 600-foot-long bridge, which was estimated to cost \$60,000 more than the other plan, which proposed a 540-foot-long bridge with higher abutments.

On November 14, 1951, DPW-HQ responded to M-H's preliminary plans by approving the 540-foot-long bridge. DPW made other preliminary design suggestions stated in part as follows:

From boring 2 (at pier 1) it would appear that the footings for the westerly pier should be established at elevation 267.0 where a resistance of 233 blows per foot 35/ was encountered in driving the casing. Elevation 270.0 which you have established for the bottom of the remaining piers, appears satisfactory when one reviews borings 3, 4, and 5 (for pier 2, 3 and 4).

35/ Blows per foot--The number of times a drop hammer hits a soil sampler, which indicates the bearing strength of the soil.

STATE OF NEW YORK  
DEPARTMENT OF PUBLIC WORKS  
DIVISION OF CONSTRUCTION

N. and  
Rm. # 3

**MOHAWK THRUWAY  
SECT. # 14**

**SUBSURFACE INFORMATION**

Name SCHANARIE CREEK BRIDGE R. H. No. None County MONTGOMERY  
 R. C. No. None U. S. C. No. None Bridge No. None  
 Weight of Hammer 25 lb Average Drop of Hammer 12"  
 Inside Diameter of Casing 2 1/4" Outside Diameter of Casing 2 7/8"  
 Inside Diameter of Spoon 1 1/2" Outside Diameter of Spoon 2" Split Type

Hole No. 4 Date AUGUST 16, 1951

DEPTH FEET	CASING			SPOON			PERCENT PASSING SIEVE NO.										NATURE OF MATERIAL
	IN	OUT	THICK	NO.	THICK	NO.	4	10	20	40	60	100	140	200	REMARKS		
0																GROUND EL. <u>276.9</u> COARSE GRAVEL & ROYALBARS	
1															Dark Gray SILT & SAND some gravel		
2															Dark Gray SILT & SAND some gr. (Compact)		
3															Gray SAND some silt some gravel		
4																	
5															Dark Gray SILT & SAND fr. gr.		
6																	
7															Light SILT some sand & gr. fr. clay		
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M = Moist  
N = Wet  
R = Refusal

PROF. EL. \_\_\_\_\_  
GROUND WATER EL. 279.1

W.C. Dickson  
Engineer in Charge

Sample shall be taken at approximately 5 ft. intervals and whenever progress of casing indicates a possible change to a different material. Wash to end of casing and take all samples dry with spoon below end of casing. If washing is inadvertently carried beyond end of casing, report under "Nature of Material". Immediately after taking, place the dry sample in the standard container furnished, label and seal to State Office promptly. Under "Nature of Material" report any delays, interruptions or obstructions encountered in driving casing or spoon. Report casing progress in blows per foot for last foot before spoon sample taken. Locate in Cross Section column wet, dry and frozen areas.

Figure 18.--Soil boring at the north end of pier 3. August 16 1951

Undoubtedly when you write the special foundation notes for this project, you will provide that once excavation has been started for a pier footing, it shall be contained until it is completed and the footing concrete poured as soon as possible thereafter, so that the material upon which the footing is to rest will not be subjected any longer than absolutely necessary to the elements and differential hydrostatic pressures. As soon as possible after the concrete footing has set up, the area between the footing concrete and sides of excavation or sheeting should be backfilled with well compacted impervious material as called for in the department's specification for Item 119. [Run of bank gravel fill]. We note that the data shows erosion in both stream banks and bed. It is suggested that you ascertain the extent of the probable erosion in the stream banks, and if your conclusions are that erosion is liable to get to or below elevation 270.0, these piers should be protected by a layer of heavy riprap 36/ around such piers.

In December 1951, the DPW-DE proposed that M-H lengthen the bridge to 595 or 775 feet to accommodate "frequent and erratic ice jams which, together with possible condition of extreme high water and runoff may create an exceedingly bad flood." According to available DPW-DE correspondence, no design discharge was calculated. Nevertheless, DPW-DE informed DPW-HQ and M-H that in 1901 there was a 50,000-cfs flood and in the spring of 1951, a high water mark of 290 feet. DPW-DE calculated that the proposed 540-foot-long bridge would raise the surface level of the creek by 1.5 feet upstream of the bridge and would increase the velocity from 7.5 to 10 fps for a flow of 43,000 cfs under conditions observed in 1951. (The high water mark was 290 feet.)

The DPW-DE proposal for lengthening the opening of the bridge was subject to DPW-HQ approval. In an unsigned memorandum dated December 20, 1951, DPW-HQ stated that there was no need to lengthen the bridge since "The banks of the river embankment and foundations at this point are proposed to be riprapped."

Detail Design.--In the summer of 1952, M-H furnished Pavlo with the preliminary drawings, which Pavlo used to prepare detail designs of the final structure. According to a Pavlo letter to NYSTA (dated October 25, 1955):

This drawing, known as "first phase", showed the type of construction desired, number of spans, typical cross sections, type of foundations, elevation of footings, allowable soil pressures and other basic design

36/ Riprap--stones, blocks of concrete, or similar protective covering material deposited upon river and streambeds and banks to prevent erosion and scour by water flow. Usually the rock is quarried and has been shaped into a rectangular prism.

requirements. Inasmuch as this preliminary design had been already reviewed and approved by DPW-HQ office, my written instructions were to follow the approved basic design in every respect in the preparation of the detailed contract plans.

The detail design included drawings and specifications for the superstructure, bearings, and substructure, and for creek bank modifications necessary to accommodate the bridge construction.

The DPW "Public Works Specifications" (DPW Specifications), 37/ dated January 2, 1951, permitted stone filling, dry riprap, run of bank gravel fill, cofferdams, and temporary steel sheet pilings along with other materials to be used in this bridge design. (See appendix D.) This document, along with Pavlo's drawings and specifications, was used by NTSB investigators to document the superstructure and substructure design features.

**Superstructure Design Features.**--Each of the five superstructure spans consisted of an 8-inch-thick reinforced concrete deck, 38/ overlaid by 4-inch-thick asphalt. The deck was supported by two main longitudinal girders, 12 feet deep, connected by transverse floor beams spaced approximately 20 feet apart. The floor beams spanned 57 feet between the main girders and had 25.5-foot tapered cantilever ends. Stringers 39/ spaced 8.5 feet apart were connected into the floor beams. According to the original drawings, ASTM A7 steel 40/ was used to fabricate the members. Members were connected with rivets.

Expansion joints separated the spans. The east end of each main girder was supported by a fixed bearing, anchored either to the concrete piers or to an abutment. The west end of each main girder was supported by a rocker (expansion) bearing. (See figures 15 and 16.)

The two main girders of each span were not attached in any way to the girders of any other span. Each span was designed to have a single unrestrained bearing or support that would be unaffected by stress from an adjacent span or structure. Bridges

37/ This book of specifications was to be used for all materials specified by designers for projects in New York State.

38/ Deck--that portion of the bridge that provides direct support for vehicular traffic, made out of reinforced concrete slab.

39/ Stringer--a longitudinal beam supporting the bridge deck, and framed into the floor beams.

40/ ASTM A7 steel--the primary structural steel used for construction prior to 1960 when A36, which had more consistent properties, was adopted. A7 had a minimum specified yield strength of 33 ksi (ASTM-American Society for Testing Materials).

so designed cannot transmit loads to other spans. This type of design, commonly known as a simply supported structure, is structurally nonredundant. 41/

During this time, M-H was designing other Thruway bridges with continuous span structures (in which spans are structurally connected to adjacent spans). An M-H employer stated that the Chief Engineer of M-H did not generally believe that a continuous span design was desirable because he was concerned with the effect of the forces on the complex design. The simple span design of the Schoharie Creek Bridge had general characteristics similar to many other bridges constructed from the late 1940s to the 1960s.

Substructure Design Features--The substructure consisted of four piers and two abutments. Each pier was supported by a spread footing on dense glacial deposits. Piers 2 and 3 were located within the main channel of Schoharie Creek, while piers 1 and 4 were located outside the main creek channel.

The two abutments were supported by batter piles 42/ and vertical piles driven through the approach embankment fill into underlying natural soil. Each abutment consisted of an end wall (or backwall) with two vertical pedestals supporting the girder bearings 43/ and wing walls. Because of the vertical grade of the bridge and adjacent roadway, the east abutment held a greater volume of soil than the west abutment. Thus, the east abutment was constructed with thicker walls, longer wing walls, and a larger number of piles.

The piers were designed as rigid concrete frames, each consisting of two columns connected by a tie beam near the column tops and supported on a plinth and spread footing. (See figure 9.) Because of the vertical grade of the bridge, height of the columns varied, from about 50 feet for pier 1 to about 60 feet for pier 4. The bottoms of the footings of piers 2, 3, and 4 were placed at an elevation of 270 feet; the bottom of pier 1 was placed at an elevation of 267 feet based on the results of the boring tests.

To help resist the tensile load in the structure, steel reinforcement bars were placed in the concrete piers. Safety Board investigators were able to review detail design calculations for determining minimal steel reinforcement needed

41/ A structurally nonredundant bridge does not offer an alternate load path to transfer loads from a failing structure to other bridge members.

42/ Batter pile--a pile driven in an inclined position to resist forces that act in other than a vertical direction.

43/ Girder bearing--the device upon which the main girder rests and transfers loads to the support pier.

for tensile loading in the tie beams, columns, and spread footings. However, no calculations were uncovered that determined the steel reinforcement needed for tensile loading in the plinth.

The tie beam, columns, and bottoms of the spread footings were reinforced by a relatively large number of steel bars with diameters up to 1.27 inches. In the top of the plinth, light steel reinforcement bars (0.5-inch-diameter spaced 18 inches apart) were used to accommodate temperature changes, as recommended by AASHTO. Each plinth was connected to the footing with steel reinforcing dowels.

The soil for each pier was to be excavated for a horizontal distance of 5 feet around the perimeter of the spread footings. Temporary steel sheet piling and cofferdams were to be used to prevent soil and water from entering the excavations for the piers. Temporary steel sheet piling (Item 83ST in DPW Specifications) was to be used at piers 1 and 4. The temporary sheeting was to be 30 feet long and was to be driven 15 feet below the bottom of the footing. However, quantity estimates called for only 5,000 square feet of temporary sheeting, an amount insufficient to completely encircle one pier. The job engineer stated that sheeting was moved from one pier to another. Sheeting was driven part of the way around pier 4, but when water began seeping into the excavations for piers 1 and 4, cofferdams were built, which required driving additional sheeting.

Cofferdams (Item 82 in the DPW Design Specifications) were to be used for piers 2 and 3. To prevent flooding of the excavation, the tops of the cofferdams were to be at an elevation of 290 feet. Sheeting in addition to that specified under Item 83ST was to be used to form the cofferdams, which were to extend downward 22 feet. The quantity estimates specified that sufficient sheeting material was to be available to allow construction of both piers at the same time (10,700 square feet).

Pavlo's design plans (figure 8) showed steel sheeting in the top view at piers 2 and 3 and not the other materials permitted by the Item 82 specification. No sheeting was shown at piers 1 and 4. The bottom view of the design plans did not show sheeting at any of the piers.

The design plans also did not specify that the sheeting was to be permanent. Item 82 of the specifications did state that the contractor was to "remove cofferdams and pumping equipment at the locations indicated on the plans or called for in the proposal in order that work may be progressed as ordered." However, the detail designer stated, in response to Safety Board questions, that he intended to leave a shield of permanent sheet piling to contain riprap and to protect the piers against erosion and scour.

The design plans indicated that the area between the spread footing and the sheeting was to be backfilled. The top view of the design plans indicated that riprap (Item 80 in the DPW Specifications) was to extend from the bottom of the footing up to the top of the "cut off" sheeting, which was the elevation at the top of the footing. From the edge of the sheeting, the riprap was to taper upward to the plinth to an elevation of 279.5 feet. The bottom view of the design plans did not show riprap to the bottom of the footing. However, the design engineer's quantity estimates were sufficient for riprap to have been placed to the bottom of the footing.

DPW Specifications indicated that the riprap thickness should be as shown on the plans. The only dimension for riprap specified on the bridge plans stated that riprap on the west abutment should be a minimum thickness of 8 inches and a maximum thickness of 15 inches. The plans also called for the riprap to be item 80 riprap.

Construction.--On February 11, 1953, DPW contracted for the construction of the section of the Thruway including the Schoharie Creek Bridge to B. Perini and Sons, Inc. (Perini). DPW authorized work by subcontractors for Perini. The three primary construction subcontractors who worked on the bridge were Monroe-Langstroth, Inc. (M-L), which built the foundations, C.L. Guild Construction, which drove the piles under the abutments, and the American Bridge Division, Co. (ABD) of U.S. Steel, which fabricated and erected the steel superstructure. 44/

Safety Board investigators interviewed a DPW engineer (inspector), Perini's job engineer, two Perini employees who worked on this project, and seven M-H employees (engineers, surveyors, and inspectors) who monitored construction for DPW and the NYSTA. Most of the M-H engineers interviewed had graduated from college shortly before this job began and had little engineering experience at the time. Photographs taken by the M-L superintendent and M-H inspectors were also obtained. Construction logs, letters, quantity pay estimates, photographs, and interviews comprise the available information on the construction of the piers.

On March 30, 1953, Perini began clearing the area and built a haul road upstream of where the bridge was to be built. The creek was diverted under the haul road through five large culverts, which directed flow between piers 2 and 3. Before beginning the excavation for the footings, Perini scraped away the top layer of stone on the river bed and built finger roads around the perimeter of the piers.

44/ M-H inspected the site daily. In addition, DPW oversaw and inspected the superstructure, the substructure, and the bearings.

Correspondence from NYSTA to the New York State Comptroller dated April 24, 1953, indicated that M-L was approved to excavate, place cofferdams and sheetings, set bar reinforcements, and place concrete for the piers. A photograph dated July 1953, shows that the plinth was completed on pier 2 and riprap tapered upward around pier 2. Pier 3 was the last pier to be built with the footing poured on September 14, 1953. (See figure 19.) The top layer of shaped stone riprap can also be seen. The job engineer stated that the riprap was probably placed between the sheeting and the pier.

Photographs taken in November or December 1953 (figure 20) show riprap on the west side of pier 2. The haul road, which according to the M-H log was to be removed during the winter, was still in place in December 1953. Neither of the photographs of the bridge show riprap around pier 3. Photographs taken during December 1953 show a large pile of stone south of the bridge between piers 3 and 4. According to these photographs, pier 4 had not been backfilled in the winter of 1953. The M-H log notes the placement of stone riprap at the Schoharie Creek Bridge pier footing on October 5, 1954, and additional riprap work at the bridge during December 1954. The M-H logs indicated there was continual work with the dry riprap during 1955, but these notations often refer specifically to the west abutment. According to the quantity estimate log, final estimates developed by M-H in February 1956 based on design plans indicated that a total of 697 cubic yards of riprap were placed at piers 2 and 3.

On October 16, 1955, the Schoharie Creek experienced its flood of record (76,500 cfs) for the period from 1900 to 1987. On October 17, 1955, the M-H log stated "slight damage to item 80 riprap Schoharie Creek Bridge south side of west abut". The log notes that on December 9, 1955, DPW phoned M-H and asked about the "condition of Item 80 around piers." On December 13, 1955, M-H responded to DPW with the requested information, but no action was noted.

Three photographs taken on October 30, 1956, show riprap at the bridge piers. These photographs show that the water level was between elevation 278 and 279 feet, which is about 1 foot below the top of the riprap as called for in the design plans. In one photograph (figure 21), very little riprap was visible on the southeast side of pier 2, while a slight mound of riprap visible on the northeast corner extends upward to an elevation of about 281 feet. The second photograph (figure 22) shows a water elevation of about 279 feet, with only a few pieces of riprap visible on the west side of pier 3. On the west side of pier 2, some riprap is visible to an elevation of about 281 feet. In the last photograph (figure 23), between the columns of pier 3 on the east side, riprap is visible at an elevation of 283, which was 3.5 feet above the height called for.



Figure 19.--Photograph of piers 1 and 2 during construction, July 1953.

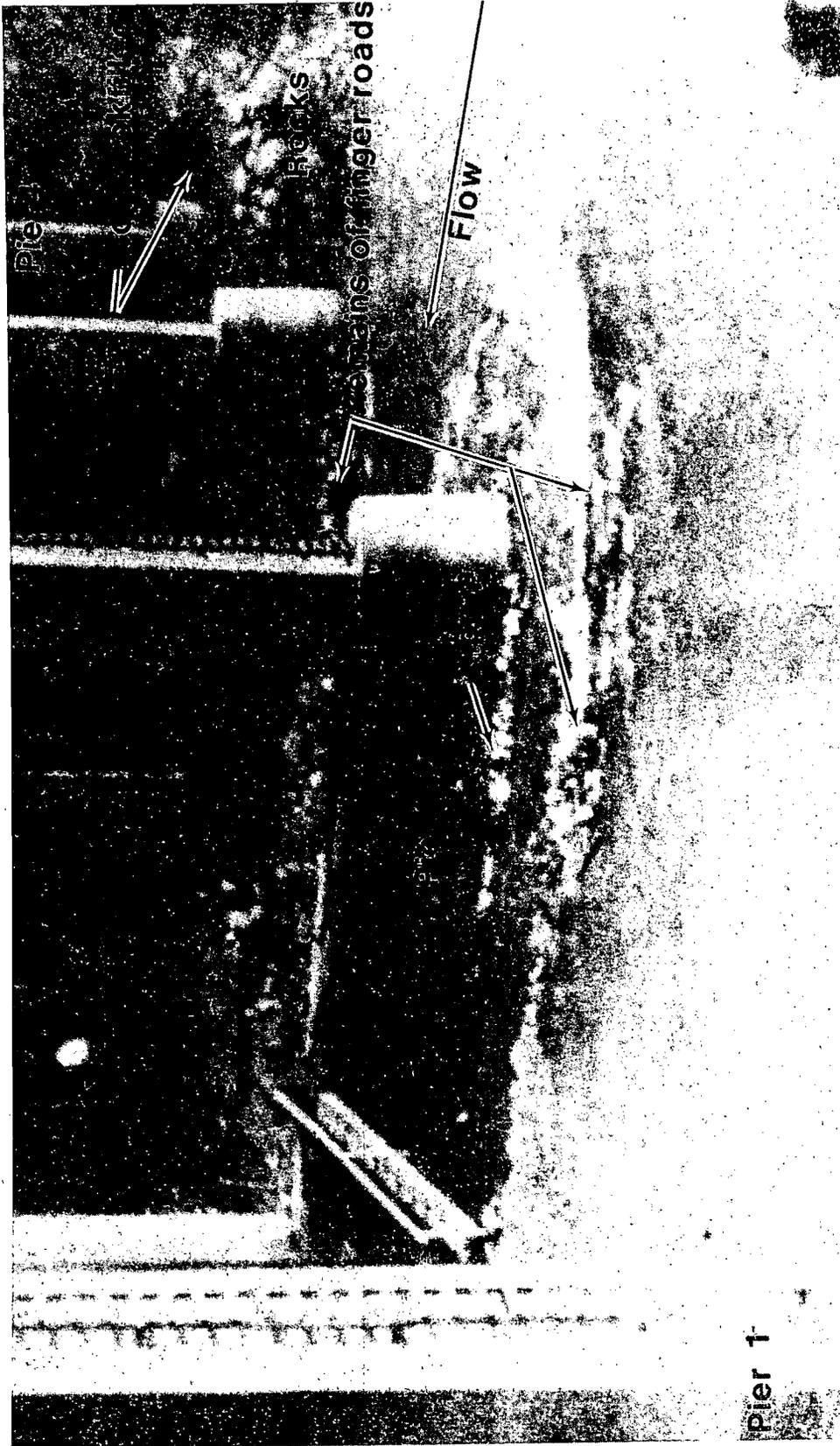


Figure 20.--Photograph of substructure construction taken during November or December 1953. Water elevation at about 280 feet.

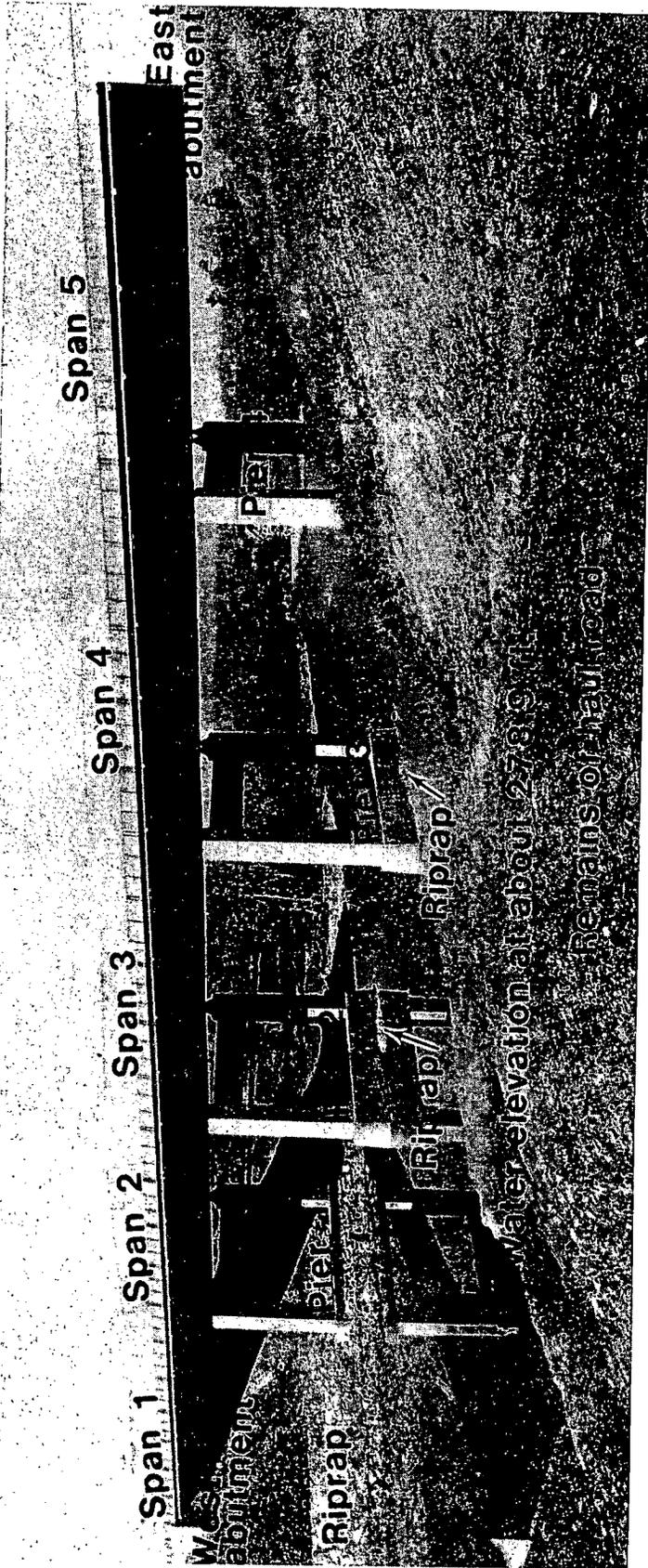


Figure 21.--Photographs of the Schoharie Creek Bridge  
Looking northwest on October 30, 1956.

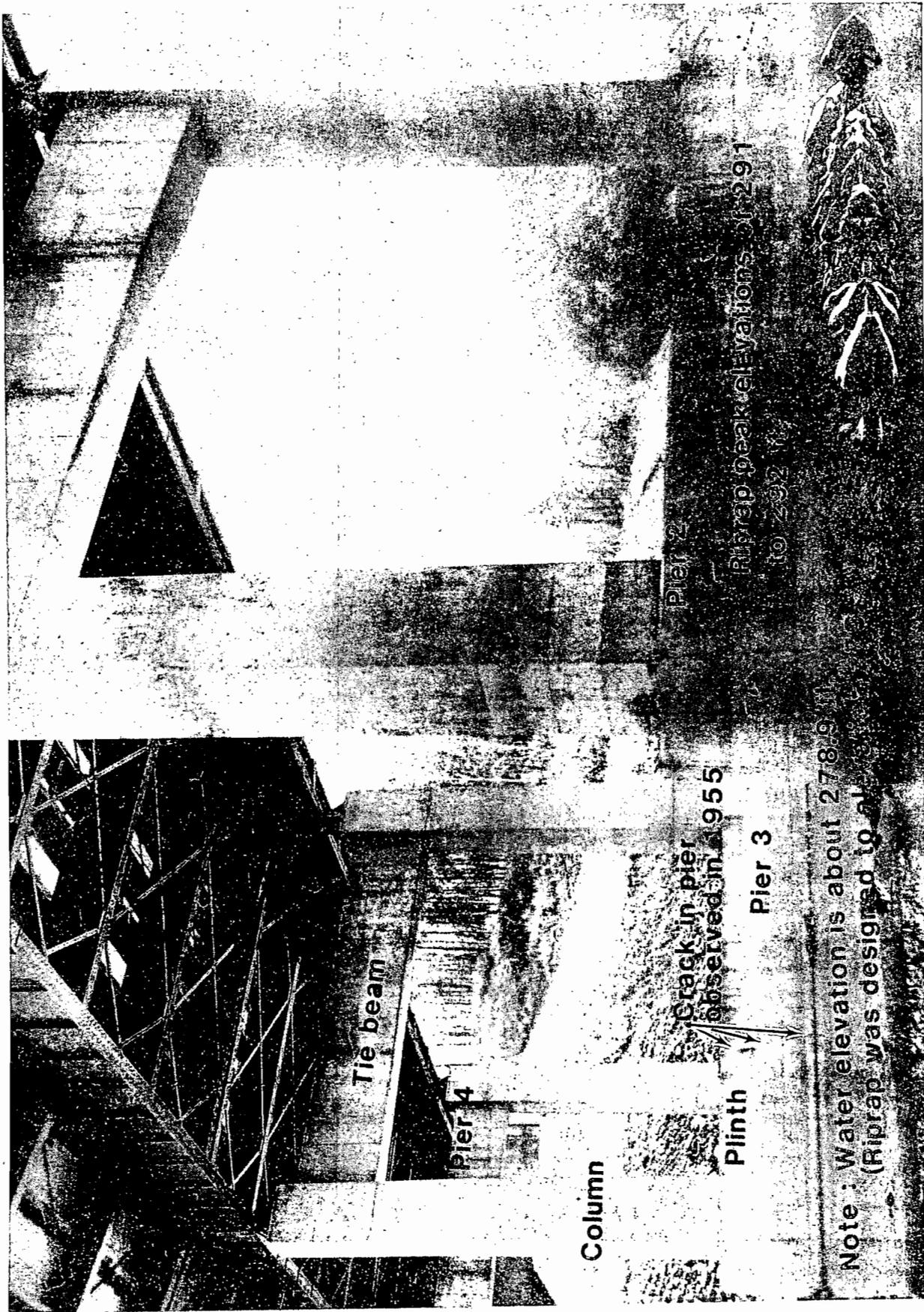


Figure 22.--Photograph of the east side of pier 3 taken on October 30, 1956.



Figure 23.--Photographs of piers 2 and 3 looking east on October 30, 1956.

The bridge was opened to traffic on October 26, 1954. However, the "punch list", the list of items of work to be finished prior to final payment, was not completed until the early part of 1956. The final inspection and acceptance of the bridge took place on May 31, 1956.

Problems.--In 1955, two major problems were observed during the bridge construction and before the flood: all plinths had cracked vertically through the middle and many of the rocker bearings were out of alignment by 0.5 to 2.4 inches. According to correspondence, at that time, the DPW-HQ was concerned that, because the bearings were highly stressed, they could become unsafe when they were 3/4 inch or more out-of-plumb. Possible solutions included installing larger bearings, welding stiffeners on the bearings, or jacking the structure and straightening the bearings. In the spring of 1956, ABD straightened the bearings after jacking the bridge spans.

Redesign.--In response to these problems, on February 28, 1956, M-H was requested to prepare a contract for additional work to the Schoharie Creek Bridge, which included changes to scuppers, abutment drainage, paving, and footings. This supplemental design contract included the construction of a 3- by 8.5- by 46-foot concrete cap with heavy steel reinforcing on the top of each plinth. The plinth reinforcement cap was to be reinforced with 53 square inches of steel bars and connected to the plinth with 44 one-inch diameter bars, each 5.75 feet long. Although the designer initially considered attaching the plinth reinforcement cap to the columns by dowels or straps, he rejected the idea out of concern that it would disturb the steel reinforcing bars in the columns.

On November 28, 1956, Hamagrael Construction Corporation was contracted to repair the bridge. The contract specified completion of all work beneath the bridge by January 20, 1957. The contract also specified that "time is the essence of this contract." The supplemental plans, which showed plinth and underlying substructure, did not show that the cracks extended down into the spread footing. However, the specifications for the repairs discussed procedures "for grouting [pressure grouting] the cracks in the existing footings in water."

### Maintenance and Repair

The maintenance of Thruway facilities is the responsibility of the Bureau of Thruway Maintenance. The Bureau is directed by the Superintendent of Thruway Maintenance, who reports to the chief engineer. One of the sections under this Bureau is the Bridge Maintenance Section, whose head is the assistant superintendent of maintenance (bridges); this section is responsible for maintaining NYSTA's bridges and for maintenance

and safety inspections. Four division engineer offices reported to the Bureau of Thruway Maintenance. Bridge inspections were done primarily by the assistant division engineer (bridges) in each division. For bridge inspections, the lines of command do not follow the formal organizational structure. In actuality, the assistant division engineers (bridges) report directly to the assistant superintendent of maintenance (bridges).

Since its construction, the Schoharie Creek Bridge was periodically maintained and repaired by the NYSTA, using its own employees or contractors. The maintenance logs indicate that above the water line, the bridge received regular maintenance. In addition, between 1981 and 1982, the bridge received a major rehabilitation, primarily because of the deterioration of the deck.

The maintenance, repair, and rehabilitation history was summarized on two sheets kept in the bridge folder (BIN folder), along with the listing of the bridge's physical features, the plans, bridge inspections, and other documents. The folder was filed in the Albany division of the NYSTA. The first entry in this history was made in 1955 and the last in 1985. The NYSTA chief engineer from 1952 to 1967 said at the Safety Board public hearing that riprap had been placed after the 1955 flood. The construction logs indicate that riprap had been moved on the south side of the west abutment and had been replaced. Entries over the years included repairing loose and missing concrete on the piers, patching and repaving the deck, and painting the superstructure. A 1968 entry concerning the bridge's footings stated, "Remove rotten conc. on Footing & Piers, replace w/Jet Crete-Did not finish." <sup>45/</sup> None of the entries mentioned riprap. The former Albany division assistant engineer (bridges) also testified during the Safety Board public hearing in June 1987 that during his years with NYSTA (1969 to 1986), he did not recall riprap ever being placed or maintained around the piers.

### Bridge Inspection

NYSTA Bridge Inspection Program. --Almost from the inception of NYSTA's maintenance organization, a routine and formal bridge inventory and inspection program has existed. The inspections have provided both structural assessment and maintenance management data. Through the late 1960s and until 1977, Thruway bridges were usually inspected annually. In 1977, the requirement for annual inspections was spelled out in Maintenance

<sup>45/</sup> The former assistant division engineer (bridges) testified that he considered the plinth and footing as synonymous. There is no evidence that any part of the footing was ever visible.

Directive Number 77.16. (See table 4 for inspections conducted between 1968 and 1986.) In October 1986, bridge inspections were extended to 2-year cycles with diver inspections of the underwater elements at 5-year intervals.

At the time of the collapse, unlike the NYSDOT inspection staff, the NYSTA inspection staff was not solely dedicated to bridge safety inspection. Their primary job has been bridge maintenance with a collateral responsibility for bridge inspections. (Until 1984, NYSDOT in-house inspections were also done by maintenance personnel.)

In 1962, a Virginia engineering firm, Byrd, Tallamy, MacDonald, and Lewis (BTM&L), conducted an extensive survey and field inspection of the Thruway bridges. The data obtained was compiled and included estimates of bridge maintenance costs for the 1962-1975 period. These inspections were done primarily for maintenance purposes and not for bridge safety inspections.

In 1971, another comprehensive inspection of all Thruway bridges was conducted by special teams of NYSTA personnel and engineers from BTM&L. This inspection was conducted to determine training and safety needs and to develop an 11-year (1972-1982) bridge maintenance and rehabilitation program. As part of the contract, the consultants developed a bridge inspection manual for the NYSTA. NYSTA did not have a bridge inspection manual before that time. The manual recommended that the substructure be inspected for scour, but did not prescribe a procedure for accomplishing this part of the inspection.

The manual included bridge inspection forms with a format different from those used previously. The new format listed 34 elements for inspection under four major categories. Under the category of substructure, riprap was listed as an element. The form required the inspector to note if an element was inspected and to estimate maintenance needs if a deficiency was noted.

For contract bridge rehabilitation projects, bridge inspections were also performed by engineering consultants working for NYSTA's Bureau of Construction and Design. These in-depth inspections were normally performed to develop contract plans and specifications. However, they were not incorporated in the biennial inspection program because the consultants were not required to fill out NYSDOT format inspection forms. As a result, some bridges were inspected by both consulting engineers and NYSTA personnel since a rehabilitation project often took several years from design through construction and the NYSTA inspection cycle would overlap the construction project. (Consultants in designing rehabilitation projects are now required to fill out the NYSDOT format inspection forms).

In 1979, all bridges on public roads (including NYSTA bridges) became subject to the National Bridge Inspection Standards (NBIS). In order to simplify documentation of bridge

Table 4.--Bridge Inspection Log for the  
Schoharie Creek Bridge

DATE	INSPECTOR'S TITLE	AGENCY	FORMAT
4/01/86	Assistant Division Engineer (Bridges)	NYSTA	NYSDOT
9/27/63	Assistant Division Engineer (Bridges)	NYSTA	NYSDOT
3/13/82	Assistant Division Engineer (Bridges)	NYSTA	NYSDOT
10/21/79	Bridge Maintenance Supervisor I	NYSTA	NYSTA
3/26/79	Consultant	NYSDOT	NYSDOT
1/78	Assistant Division Engineer (Bridges)	NYSTA	NYSTA
11/14/78	Engineering Technician	NYSTA	NYSTA
1977	Consultant	NYSTA	In-Depth Report
10/76	Assistant Division Engineer (Bridges)	NYSTA	NYSTA
12/4/75	Bridge Maintenance Supervisor I	NYSTA	NYSTA
10/10/74	Bridge Maintenance Supervisor II	NYSTA	NYSTA
12/7/73	Bridge Maintenance Supervisor I	NYSTA	NYSTA
8/72	Assistant Division Engineer (Bridges)	NYSTA	NYSTA
6/16/71	Assistant Civil Engineer	NYSTA	Consultant
11/16/70	Assistant Civil Engineer	NYSTA	NYSTA
12/12/69	UNKNOWN	NYSTA	NYSTA
11/19/68	UNKNOWN	NYSTA	NYSTA

conditions, the NYSTA adopted NYSDOT's inspection forms. These forms required more information than the NYSTA's previous forms and as a result, more accurately portrayed the structural condition of the bridge.

Schoharie Creek Bridge Inspections.--Inspection reports on file in the NYSTA's Albany division revealed that the Schoharie Creek Bridge had been inspected annually or biennially since 1968. The Safety Board could not determine how many bridge inspections were conducted before 1968. The bridge was inspected a total of 16 times by the Albany division of the NYSTA with the last inspection on April 1, 1986. These inspections of the bridge were conducted above water.

In 1972, 1973, 1974, and 1979 the inspection reports indicated that riprap was inspected, and that no deficiencies were noted. The 1974 inspection report indicated that the footing under pier 3 was spalled on the ends. Safety Board staff asked the inspector if he was able to see the footing. He stated that his comment actually referred to the condition of the plinth. He also stated that the riprap looked much as it had when he inspected it previously, and that it appeared to be in good condition. However, he did state that he had never taken measurements in the channel or around the piers.

The Albany assistant division engineer (bridges) inspected the bridge in 1972 and three other times in the seventies, namely, in 1970, 1976, and 1978. At other times, these inspections were conducted by one of his subordinates or by a consulting engineer.

In 1977, the NYSTA contracted with a consulting firm to provide design engineering services for the rehabilitation of the Schoharie Creek Bridge. The contract was to consist of two phases: (1) a field inspection, survey, and report, and (2) preparation of construction plans for the rehabilitation of the bridge. The inspection and rehabilitation plans are further discussed in a later section. (The rehabilitation was completed in 1982.)

On March 26, 1979, the bridge was inspected by a consulting firm hired by the NYSDOT. This inspection was conducted to comply with the new Federal law requirements that all bridges on public roads were to be inspected, inventoried, and appraised in accordance with the National Bridge Inspection Standards. This inspection will be discussed in greater detail in a later section of the report.

Between the 1982 rehabilitation of the bridge and the collapse, the Albany assistant division engineer (bridges) inspected it twice, in 1983 and again in 1986. The inspections followed the format outlined in the NYSDOT Bridge Inspection Manual 82 (NYSDOT - BIM - 82), which explains the documentation requirements for general bridge inspections in New York State.

Documentation consists of assigning numerical condition ratings to the various bridge elements on forms supplemented by required notes, sketches, photographs, and scour documentation. The rating system assesses the individual bridge elements on a scale from 1 to 9. The following ratings were listed in the Inspection User's manual for the New York State:

- 1 - Potentially Hazardous
- 2 - Used to shade between a rate of 1 and 3
- 3 - Serious deterioration or not functioning as originally designed
- 4 - Used to shade between a rate of 3 and 4
- 5 - Minor deterioration and is functioning as originally designed
- 6 - Used to shade between 5 and 7
- 7 - New condition
- 8 - Not applicable
- 9 - Unknown

The pier ratings given to three of the elements were:

	<u>Footings</u>		<u>Erosion or Scour</u>		<u>Piles</u>	
	1983	1986	1983	1986	1983	1986
Pier 1	6	5	5	5	9	9
Pier 2	6	5	6	9	9	9
Pier 3	6	5	6	9	9	9
Pier 4	6	6	5	5	9	9

The NYSDOT BIM-82 states that when rating footings, "a rating of 5 should be used if a footing has minor deterioration or has undergone movement causing minor distress or movement to the pier or superstructure." This rating also indicated that the pier was functioning as it was originally designed. A rating of 7 is suggested "...when the footings are visible and in excellent condition." A rating of 6 is used to shade between 5 and 7. A rating of 9 means the condition of the footing is unknown.

For rating erosion or scour, NYSDOT BIM-82 states that "If an erosion or scour problem only affects material above the bottom of footing, then the rating should be in the 3 to 6 range.

For rating piles, NYSDOT BIM-82 requires that a rating of 9 be given when the condition is "unknown"; the manual states that this is the most commonly used rating. A rating of 8 is to be given if the inspector knows there are no piles under the piers.

In 1983 and 1986, the bridge was given an overall rating of 5, which indicated that the primary structural members were in relatively good condition. Neither inspection report contained any documentation on scour.

A month after the 1986 inspection, the assistant superintendent of thruway maintenance (bridges) questioned the ratings given the piers in the 1986 report, which were poorer than the ratings given in the 1983 inspection. However, after a visit to the bridge with the assistant division engineer (bridges), he agreed that the reduced ratings were warranted.

Since the Safety Board's public hearing, the present assistant superintendent for maintenance (bridges) has pointed out that the NYSDOT Highway Maintenance Guidelines state, in part, "Repairs should be made, [using heavy stonefill or riprap] before scour progresses to a depth dangerous to the stability of a structure (1/2 of the thickness of pier footing)". He stated that during his limited observation during his visit to the site 1 month after the 1986 inspection, "...no portions of the footings were visible let alone 1/2 of their depth." He further pointed out that "probing" was not in the NYSDOT maintenance guidelines.

At no time since its construction had the bridge received an underwater inspection of its foundation and underwater elements. The first detailed underwater inspection of the bridge was scheduled for 1987, but the collapse occurred before the inspection took place.

NYSDOT Bridge Inspection Program--Under NYSDOT's comprehensive bridge inspection program, all State-owned and maintained bridges are inspected by in-house teams located in the State's transportation regions. All local bridges (village, town, and county) and all bridges in New York City are inspected by consulting engineers under contract by NYSDOT. Bridges on systems owned and operated by public authorities and commissions are generally inspected by these entities and not by the NYSDOT.

The Structures Design and Construction Division (Bridge Inspection Unit) in the main office provides overall guidance to the regional inspection teams. This division has primary responsibility for collecting bridge data, which includes the overall management of the structures inventory and inspection system. They supply the FHWA with data for the national inventory and appraisal system.

The NYSTA receives bridge inspection material from the NYSDOT and uses NYSDOT forms in inspecting their bridges. The NYSTA has sent some of their bridge inspection personnel to training courses sponsored by the NYSDOT.

There is, however, no formal communication between the NYSDOT region's in-house inspection units and the NYSTA assistant division engineers (bridges). NYSTA forwards its bridge inspection reports directly to the NYSDOT main office, which reviews the reports and then forwards appropriate copies to the NYSDOT region in which the bridge is located. Additionally, the NYSDOT main office inputs bridge inspection data received from the NYSTA onto a computer tape and periodically forwards the information to the FHWA for the national inventory and appraisal system. Although the NYSTA and other toll authorities in the State inspect their bridges, none of the authorities are under the legal obligation to report the results to the NYSDOT. State legislation, proposed since the collapse of the Schoharie Creek Bridge, would mandate that the authorities inspect all bridges under their jurisdiction in accordance with the standards established by both the Federal Highway Administration (FHWA) and the NYSDOT and report the results to the NYSDOT.

NYSDOT'S 1979 Inspection.--The 1978 Surface Transportation Assistance Act mandated that bridges not on the Federal-aid system but on public roads (also known as off-system bridges) be inspected, inventoried, and appraised in accordance with the NBIS. According to NYSDOT officials, the concern was that these bridges, which included the Schoharie Creek Bridge, had not been inspected sufficiently to meet the NBIS.

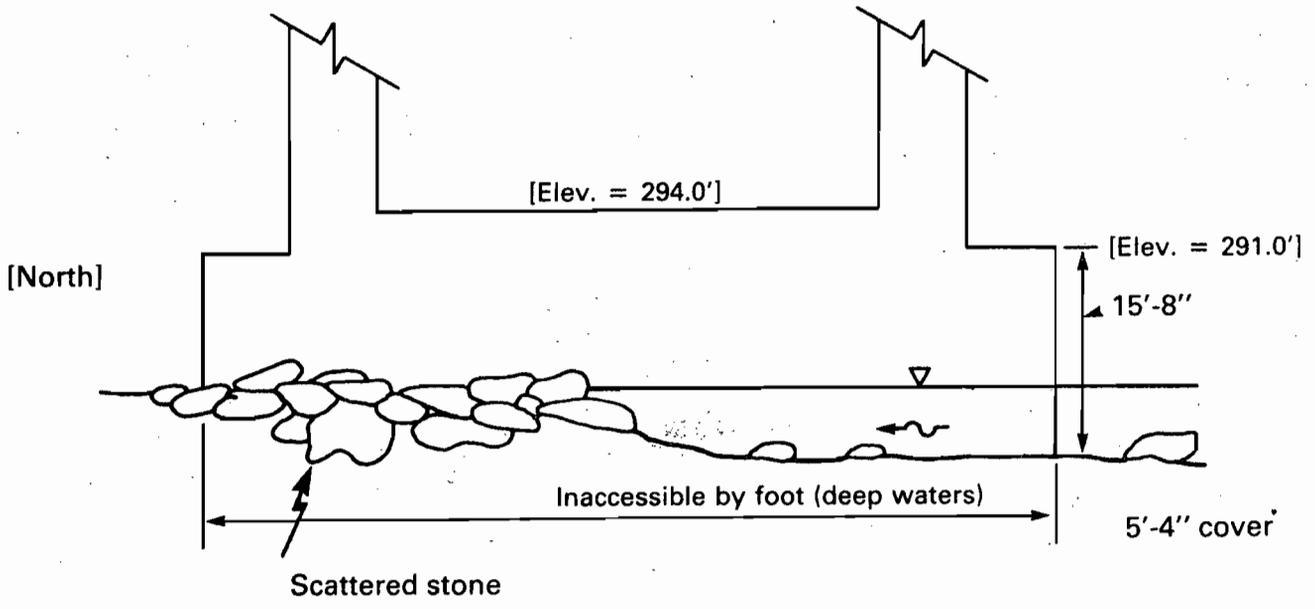
NYSDOT negotiated contracts with 22 engineering firms to perform the inspections. The inspection of the Schoharie Creek Bridge was awarded to Seelye, Stevenson, Value, and Knecht, which conducted an initial inspection of the Schoharie Creek Bridge on March 26, 1979. The team leader (since deceased) of the three-man inspection team had a New York State Professional Engineer's License and at least 3 years experience in bridge design, inspection, and other bridge-related tasks, thus qualifying him for the position in accordance with the NBIS. He was accompanied by a technician and an assistant team leader, who had a degree in civil engineering.

Because of the high level and velocity of the water on March 26, 1979, no drop line readings 46/ were made at the piers. On August 15, 1979, after the water flow had diminished, the assistant team leader returned to the bridge to document the streambed around the piers. Copies of the sketches are shown in figures 24 and 25.

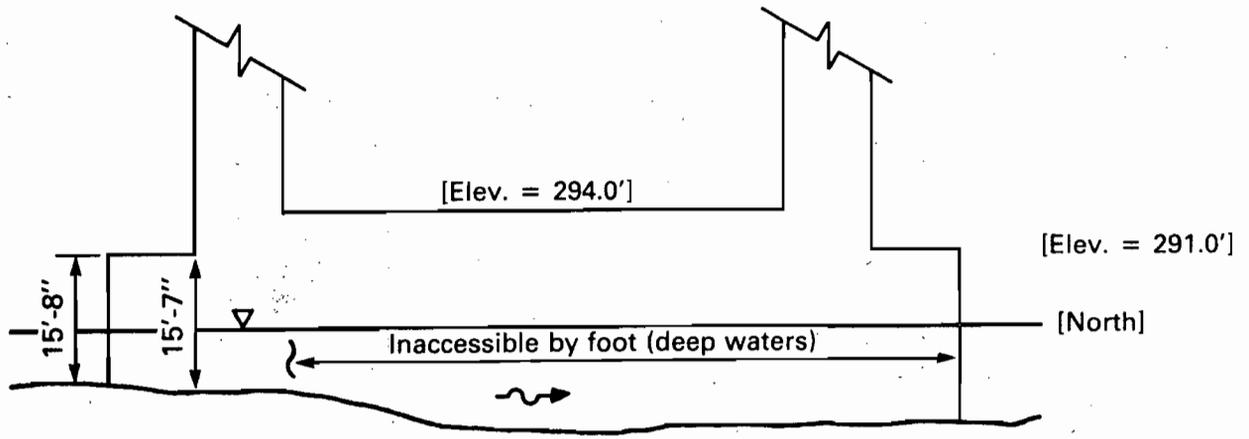
The sketches indicated that pier 2 (figure 24) had 4 inches of cover 47/ above the top of footing on the south (upstream) end, but very few rocks were shown. This amount of cover continued along the west face for 1/3 the length of the footing and then increased. Scattered stones touching each other are shown from the middle of the footing to the north end of the footing. Along the east face, the amount of cover was also 4

46/ Measurements are taken from some reference point on the bridge to the streambed with a string and weight.

47/ Cover, as used in this report, is material (which may be soil, gravel, stone, etc.) above the top of the footing.



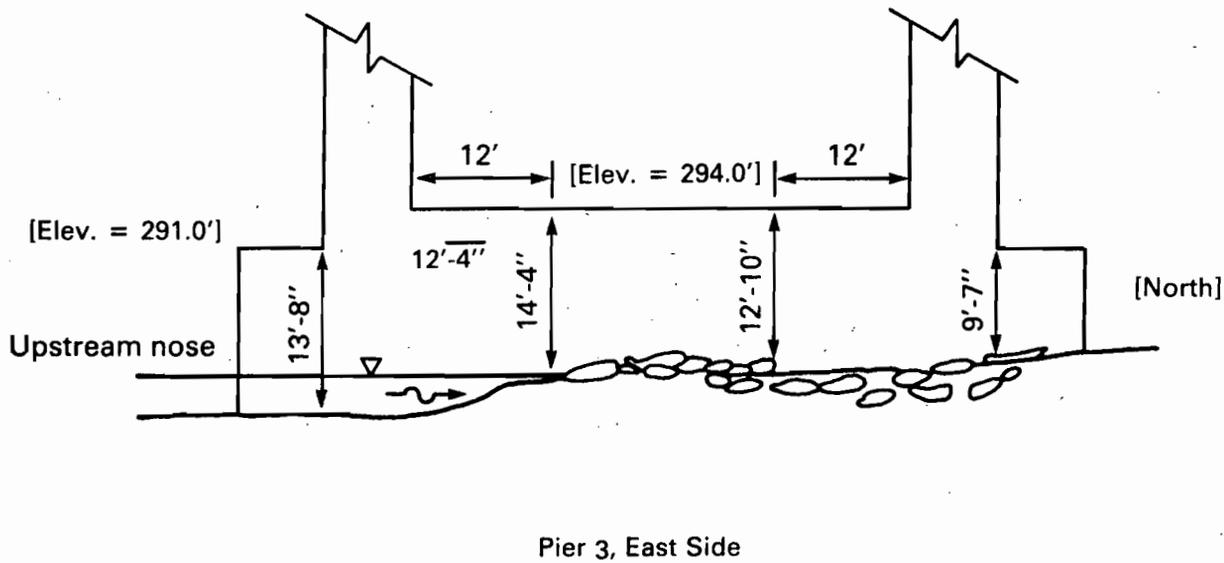
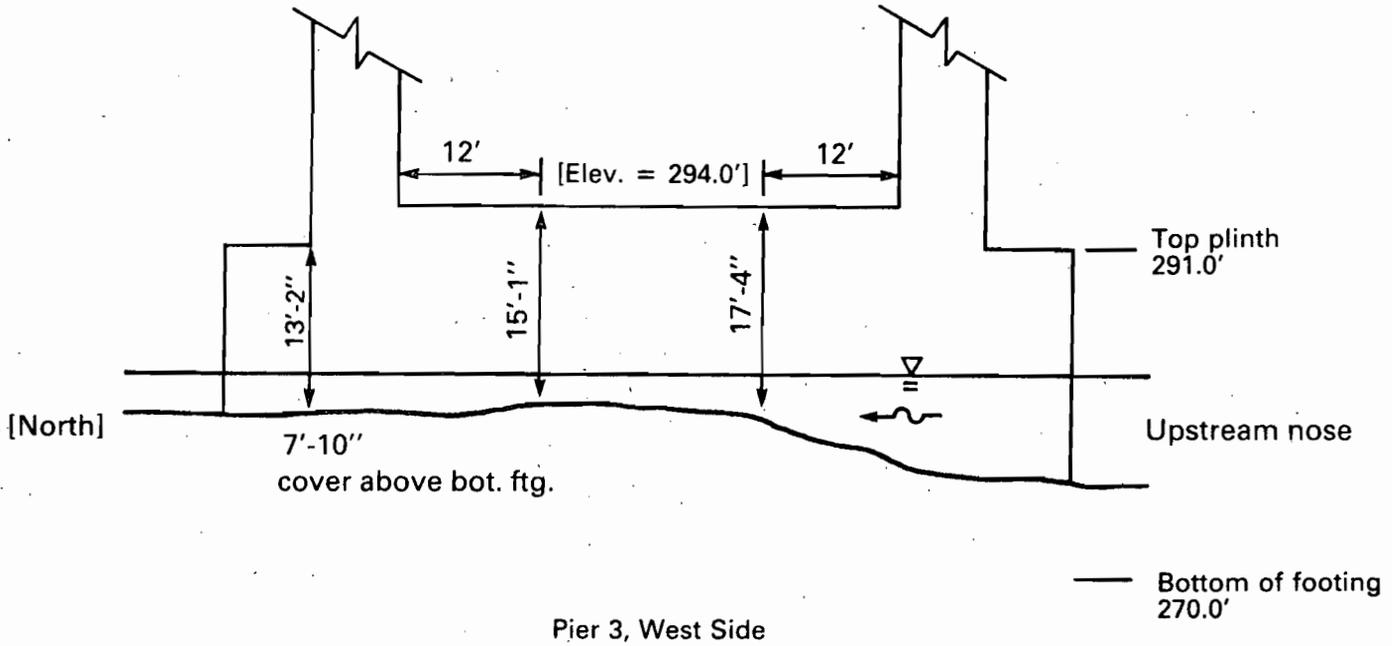
Pier 2, West Side



Pier 2, East Side

NOTE: Bracketed material was added by NTSB for clarification

Figure 24.--Seelye sketches of pier 2.



NOTE: Bracketed material was added by NTSB for clarification.

Figure 25.--Seelye sketches of pier 3.

inches at the upstream nose and remained at this level for about 10 feet north of the nose. About 10 feet north of the upstream nose, the cover decreased. No dimensions were given, but a note stated "inaccessible by foot deep water." (The assistant team leader stated that "inaccessible by foot" meant that the water depth was 3 feet or more). No stones were shown on the east face.

On pier 3 (figure 25), on the east face, about 5 feet from the upstream nose, the sketch showed 2 feet 4 inches of cover (no stones shown) that stayed fairly constant to a point 24 feet north of the nose, where 4 to 8 inches of cover was shown. The stone cover then varied from 6 to 7 feet to the north end of the footing. On the west face, from the upstream nose to a distance almost 24 feet downstream, there were no measurements, again with the note "inaccessible by foot". The streambed, however, was shown in the sketch to be lower than it was at a point 24 feet north of the nose, which had 1 foot 8 inches of cover. At a point 24 feet south of the downstream nose, the cover was 3 feet 11 inches. Five feet south of the nose, the cover was 2 feet 10 inches. No stones were shown.

On the bridge inspection and condition report, dated March 26, 1979, elements under "Stream Channel" were rated as follows:

erosion and scour	6
channel siltation	6
bank protection	8
recommendation	6

Ratings for three elements of the piers were as follows:

	<u>Footings</u>	<u>Erosion or Scour</u>	<u>Piles</u>
Pier 1	9	6	8
Pier 2	9	6	8
Pier 3	9	6	8
Pier 4	9	6	8

Under remarks, the team leader wrote, "Scour category is 2B." According to a NYSDOT publication, this rating indicated that scour measurements were to be obtained by drop line readings.

The agreement with Seelye, Stevenson, Value, and Knecht for bridge inspection services stated that if the engineer observed a condition that required immediate attention, he was to call NYSDOT's project manager and follow this with a letter. The NYSDOT project manager was then to notify the owner of the bridge. The project manager indicated that no such notification was received for the inspection of the Schoharie Creek Bridge.

NYSDOT selected for review between 10 and 25 percent of the bridge inspection reports it received; the reports were reviewed primarily to check format and coding. Copies of the reports were sent to the owners of the bridge. The Safety Board could not determine if NYSTA or NYSDOT personnel reviewed the consultant's inspection report for the Schoharie Creek Bridge.

Bridge Inspection Standards and Federal Oversight.--In December 1967, the Silver Bridge at Point Pleasant, West Virginia, collapsed and killed 46 persons. The Safety Board's investigation of this accident revealed that many States did not have formal bridge inspection programs or systems for keeping bridge inventory information. In response to this tragedy, the U.S. Congress enacted the Federal Aid Highway Act of 1968, which directed the Secretary of the Department of Transportation to establish national bridge inspection standards.

As a result of this legislation, the NBIS became effective in April 1971. The NBIS required the inspection of all public bridges on the Federal-aid system. The Surface Transportation Act of 1978 extended the requirements of NBIS to all bridges carrying public roads, even if they were not part of the Federal-aid system. The FHWA administers the program.

The NBIS requires all bridges to be inspected at 2-year intervals, but the individual States can decide the level and detail of inspections for particular elements of the bridges. Inspections are to be made in accordance with the "Manual for Maintenance Inspection of Bridges" 48/ published by AASHTO.

The NBIS 49/ stipulates that: (1) each highway department have a bridge inspection organization, (2) bridge inspectors meet minimum qualifications, (3) each structure be rated as to its safe load carrying capacity, and (4) inspection records and bridge inventories be prepared and maintained in accordance with the NBIS. The depth and frequency of inspections is to depend on such factors as age, traffic characteristics, state of maintenance, and known deficiencies. The evaluation of these factors is the responsibility of the individual in charge of the inspection program. The standards do not require that either the individual in charge of bridge inspections or the team leader be tested on bridge inspection procedures. However, to qualify for these positions, individuals must be registered as professional engineers, qualify for registration, or have a specified minimum number of years of experience and have completed a comprehensive course based on the FHWA's "Bridge Inspector's Training Manual." 50/

48/ First edition published in July 1970. Subsequent editions have been printed in 1974, 1979, and 1984.

49/ See appendix E for the complete NBIS as promulgated in 23 CFR Part 650, Subpart C.

50/ The "Bridge Inspector's Training Manual" was developed by a joint Federal-State task force and published in 1970.

The "Manual for Maintenance Inspection of Bridges" states that the individual in charge of the unit responsible for bridge inspections:

...must be thoroughly familiar with design and construction features of the bridge to properly interpret what is observed and reported, ... be able to recognize any structural deficiency, assess its seriousness, and take appropriate action necessary to keep the bridge in a safe condition...recognize areas of the bridge where a problem is incipient so that preventive maintenance can be properly programmed.

Under the section entitled, "Piers and Abutments," it further states:

Investigate the footings for evidence of significant scour or undercutting...Probing and/or diving will be necessary at many piers.... Particular attention should be given to foundations on spread footings where scour or erosion can be much more critical than a foundation on piles. However, be aware that scour and undercutting of a pier on piles...can also be quite serious.

The FHWA reviews State bridge inspection programs to determine if a State is meeting inspection frequency, has qualified inspectors, and if the programs comply with the bridge inspection and inventory requirements. The reviews are conducted on three levels: (a) the FHWA division office conducts an annual review of the State's bridge inspection program and discusses with State officials the State's compliance with the minimum requirements of the NBIS; (b) the FHWA regional office conducts additional reviews to determine if the division office audit is reasonable and sound; and (c) the FHWA headquarters office in Washington, D.C., reviews regional programs and provides overall administration of the NBIS.

Over the years, FHWA headquarters has directed its division offices to review the conditions in each State to identify those governmental entities that have not complied with the NBIS. In 1986, FHWA conducted a management review of the NYSDOT bridge inspection program, which encompassed the NYSTA inspection program. (The FHWA division has never directly reviewed the bridge inspection program of the NYSTA.) The FHWA noted that the NYSTA did not meet the inspection frequency on 50 percent of their bridges. The FHWA also expressed concern that the NYSDOT underwater inspection program was behind schedule.

In November 1987, the FHWA conducted a field survey of all States and found that about 43,000 bridges had overdue inspections. The overdue inspections were 7.5 percent of the 577,000 bridges in the nation. According to NYSDOT, FHWA data on inspections of bridges within New York indicate that in 1986 and 1987, about 5 to 6 percent of the bridges had not been inspected for more than 2 years.

Before 1985, FHWA had not emphasized underwater inspections or required FHWA divisions to review the State's underwater inspection capabilities in their annual reviews of the bridge inspection program. As a result of the Safety Board's investigation into the collapse of the Chickasawbogue Bridge in April 1985, <sup>51/</sup> in June 1985, FHWA required each State to develop such a program. Each State was to have (as a minimum): (1) written criteria as to when underwater inspection was required, (2) a list of bridges in need of underwater inspections and the frequency needed, (3) method of underwater inspection, and (4) specific records of inspection results and followup to any identified major deficiencies. On April 9, 1986, an FHWA memorandum to each regional administrator stated that if a State did not have an underwater inspection program, the State was not complying with the NBIS.

### Bridge Rehabilitation

On August 1, 1977, the NYSTA contracted with Dale Engineering, Inc. (Dale), a Utica-based consulting engineering firm to provide a field inspection, survey, and report for both the superstructure and substructure of the Schoharie Creek Bridge (phase I of the contract). The substructure investigation included only those portions of the abutments, wingwalls, and piers that were above ground or water. If the firm found or suspected abnormal movements of the structure, they were to determine the nature or cause of the movements and advise the NYSTA of any conditions requiring immediate remedial action. Dale's primary contact with the NYSTA was the head of the design unit, who reported directly to the director of construction and design. Phase II of the contract was the preparation of construction plans for a rehabilitation contract.

During testimony given at the Safety Board's public hearing, the project manager for the firm stated that in August 1977, they took some drop line readings around the plinth. The water level was extremely low and the water was clear. They compared the readings with original as-built drawings and determined that some of the riprap was no longer present. The firm could not find the actual drop line readings in their files, but the project manager stated that the profile and elevation drawings, dated December 28, 1979, were prepared for the NYSTA based on drop line measurements. The drawings show both the firm's and the NYSTA's logos. Copies of a photograph and drawings from the design plans are shown in figures 26 and 27.

<sup>51/</sup> For more information, see Highway Accident Report--"Collapse of the U.S. 43 Chickasawbogue Bridge Spans Near Mobile, Alabama, April 24, 1985" (NTSB/HAR-86/01).

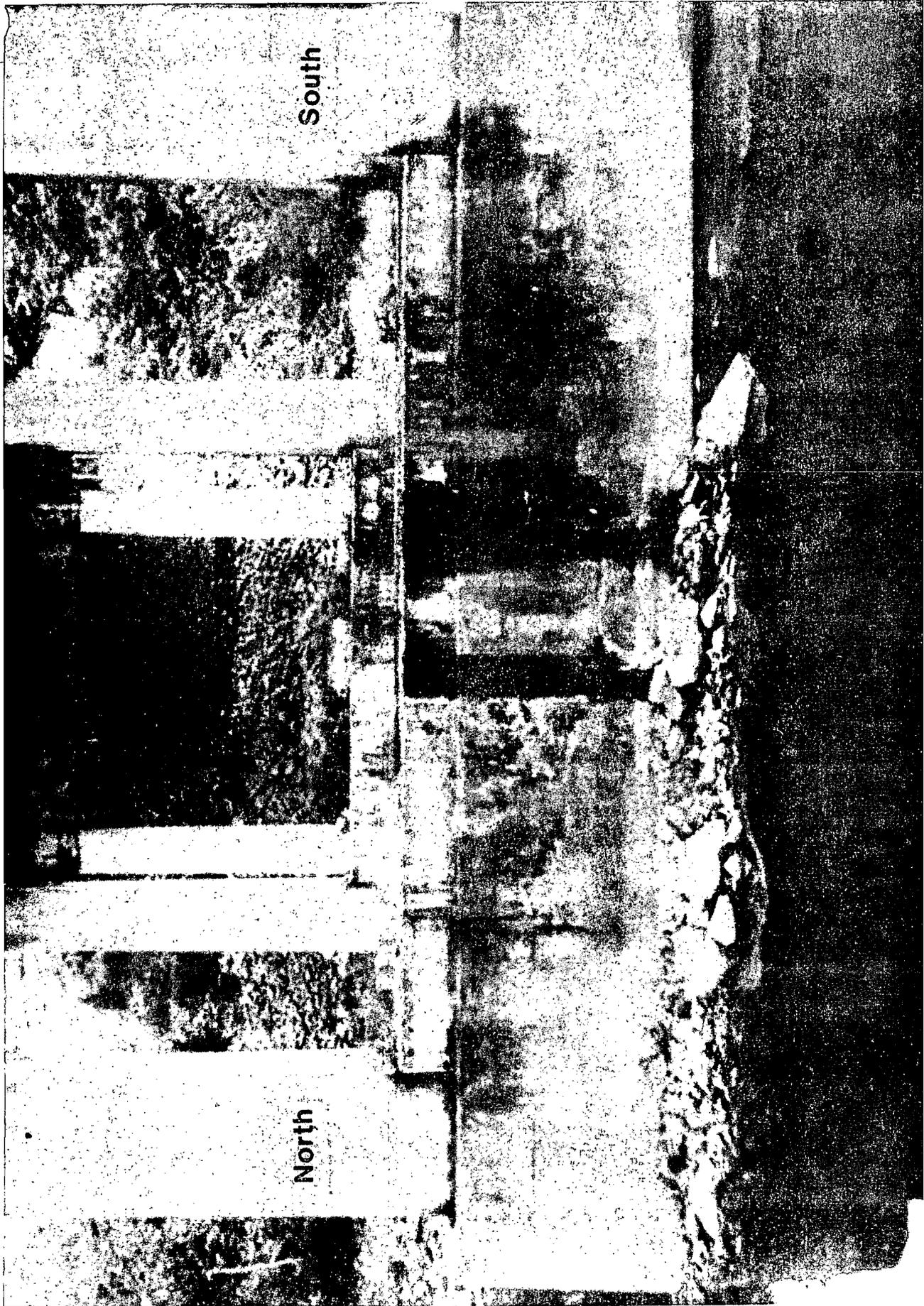
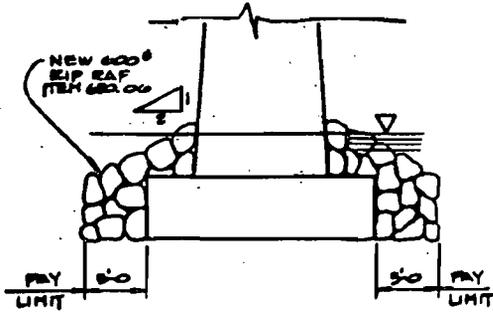
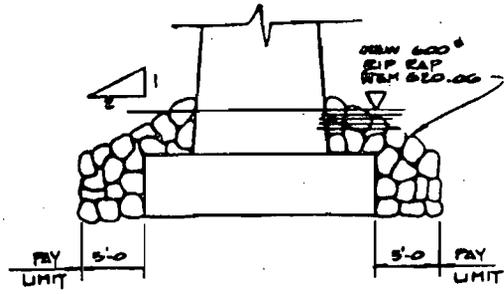


Figure 26.--West side of pier 2, August 1977  
(water elevation 278-279).

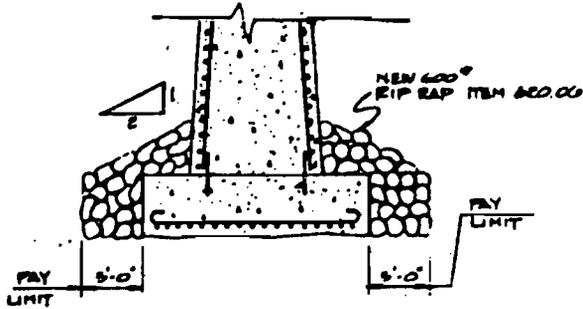


North  
Elevation

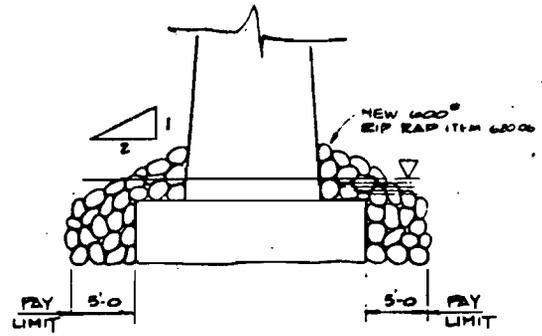


South  
Elevation

Pier #2



North  
Elevation



South  
Elevation

Pier #3

(Pier 1 not shown because it was not involved in collapse)

Figure 27.--Design drawings prepared by Dale Engineering.

The following elevations were determined from depths measured along the south elevation of the bridge and from the top of the plinth. (Note: Scaling a print is subject to inherent inaccuracies in the drawings and measurements.)

	Pier 2 streambed elevation	Pier 3 streambed elevation	Water elevation
West Side	276'	278'	278'
East Side	276'	279'	278'

NOTE: Top of the original plinth (elevation = 291.0') to top of footing (elevation = 275.0') is 16 feet. Top of the original plinth to top of riprap (design) is about 11 1/2 feet (riprap elevation = 279.5').

The detail drawings for pier 3 also show existing streambed and water elevations. On the north end of the west side, the existing streambed and water elevation was scaled at 14 feet (elevation = 277') below the top of the plinth. The east side showed the existing streambed elevation at 10 feet (elevation = 281') below the top of the plinth. On the south end of the east side, the existing streambed was 12 feet (elevation = 279') below the top of the plinth.

As part of phase I of the contract, the consultants prepared two reports. The first was titled, "Existing Conditions Survey Report of Schoharie Creek Bridge" (undated). An internal memo indicates the NYSTA received this report before May 19, 1978. The second report was entitled, "Preliminary Plans and Report." An internal NYSTA memo indicates NYSTA received this report before August 18, 1978.

The two reports contain some of the same information. However, the first report stated:

Pier No. 2, which is out in the Schoharie Creek, is in fairly good shape except for some minor scouring around the pier. It does not appear to have undermined it.

Pier No. 3 has some scour on the upstream end and has undermined some of the gunite 52/ and repair work.

The report made seven recommendations, which the consultants felt were "...the minimum to bring the structure back to an 'as new' condition." Part of the third recommendation stated:

52/ Gunite--pressure-sprayed concrete.

...place additional riprap around the three piers that are exposed to [the water flow in] the creek, especially during high water.

The second report stated that:

The creek has started to erode the riprap protection around the piers.

New riprap should be installed around three of the piers to prevent further erosion which would result in underpinning [undermining] the spread footings of the piers.

The report contained a preliminary engineer's estimate for items necessary in the rehabilitation of the bridge. Item 620.06 called for 800 cubic yards of dry riprap at a cost of \$20 per cubic yard. The total rehabilitation contract was estimated to cost \$2,732,000.

The NYSTA received the final plans, specifications, and estimates for the rehabilitation on February 1, 1980. The estimate for dry riprap was 600 cubic yards. On the Dale drawings for piers 1, 2, and 3, new 600-pound riprap was shown from the bottom of the footing up to 4 1/2 feet above the top of footing, and sloping to a point 5 feet away from the perimeter of the footing. (See figure 27.) The dimensions were similar to those specified on the original design plans. The NYSTA reviewed the plans and noted that "Cofferdam required for riprap proposed at Piers 1, 2 and 3."

In 1980, an employee of the NYSTA in the Bureau of Construction and Design was given the job of finalizing the plans for rehabilitating this bridge along with those of an adjacent bridge submitted by another consultant. The rehabilitation designs were to be combined into one contract. The employee was neither a graduate engineer nor a licensed professional engineer. However, he reported to a section head who was both.

The employee stated that he visited the Schoharie Creek Bridge and observed the existing conditions, including the riprap. He indicated that he did not measure the depth of the riprap. The water was low and he could walk around piers 1, 3 and 4. He did not see any scour holes or depressions around the piers.

He returned to the office and directed a draftsman to remove all references to new riprap on the plans. He stated during an interview with Safety Board investigators that he had not consulted anyone else regarding his decision. The item for riprap was also removed from the estimate of quantities.

The employee had a copy of the consultant's engineering reports. He recalled that the report recommended riprap but he did not feel there was a substantial need for it. He was not given the BIN folder (bridge inspection and maintenance files) but the folder was available to him. However, when questioned about his decision, he indicated that he may have looked into previous inspection reports.

The contract for the rehabilitation of the Schoharie Creek Bridge and the adjacent bridge was let on February 4, 1981. The work was to be completed by July 30, 1982. Manuel Elkin, a consulting engineering firm from New York City, New York, provided inspection services during the project. NYSTA engineering personnel visited the site as necessary during construction but did not provide any comments about the riprap.

#### Overview by NYSTA's Insurance Carrier

The Schoharie Creek Bridge, along with other facilities owned by the NYSTA, was insured against loss. Insurance companies require from the owner certain information about the facilities to be insured. The companies review the insured facilities and, in the case of bridges, the bridge inspection policies of the owners.

On November 7, 1985, a senior construction specialist (a licensed professional engineer) from the insurer visited the NYSTA's headquarters office for underwriting purposes. He met with the assistant superintendent of maintenance (bridges). As a result of the visit, the construction specialist made 11 recommendations in a letter dated November 26, 1985. Three of the recommendations involved the substructure:

#### Recommendation 85-1

All substructures that are hidden from view by water should be inspected by divers. This is necessary to determine if unseen corrosion damage or scour might have occurred. These inspections should be made periodically on a 3-5 year schedule, the first inspection should be performed within 6 months on bridges not already inspected.

#### Recommendation 85-6

Channel and scour information should be obtained according to procedures listed in Section 12 of the BRIDGE INSPECTION MANUAL -82 (or later edition) for each water crossing. Documentation should be as provided in the manual to allow replication of the survey at future inspections.

Recommendation 85-8

Sounding devices, rods or the means to allow proper measurement of scour and channel configuration should also be available in each region for use when required.

Another recommendation, 85-9, related to quality control. It stated, in part:

Quality control of inspections as provided by the BRIDGE INSPECTION MANUAL - 82 (or later edition) should be strictly adhered to. A 100% office review of bridge inspection should be taken to determine what information should be requested of regions to bring inspection files to the quality demanded in the BRIDGE INSPECTION MANUAL.

The NYSTA (the assistant chief engineer) answered the insurance carrier on January 2, 1986. In response to Recommendation 85-1 on underwater inspection, he wrote that an analysis of the bridge inventory was underway "to identify structures requiring underwater inspections and the date of last inspection." Recommendation 85-6, Channel and Scour Information, would be reviewed for the 1986 series of inspections, and any deficiencies remedied. For Recommendation 85-8, he implied that the NYSTA was properly equipped to do bridge inspections. He attached NYSTA's current bridge inspection equipment list. One of the items on the list was probing rods.

In response to Recommendation 85-9 on quality control of inspections, he indicated that all inspections were receiving an "office review" and that the NYSDOT reviewed the inspections. He further stated that the NYSTA would "attempt to comply with the latest manual during each biennial bridge inspection," placing additional emphasis on the office review and elimination of deficiencies.

In December 1985, as a result of Recommendation 85-1, the NYSTA proposed that bridge substructures hidden from view by water be inspected with the assistance of a consultant and/or contractor. During this period, NYSDOT was also proposing underwater inspections for many of their bridges.

Early in 1986, the NYSDOT proposed to expand their underwater inspection program to include Thruway bridges that needed underwater inspections. The NYSTA concluded that this was the fastest and most economical way to get their bridges inspected. The Schoharie Creek Bridge was scheduled to be included in the proposed inspections. The selection of the Schoharie Creek Bridge was the result of the Albany assistant division engineer's (bridges) recommendation. In answering a query from the assistant superintendent of maintenance (bridges), he stated that pier 2 required underwater inspection. His

inspection. His comments were that "footing of pier 2 on occasion, is subjected to very turbulent debris filled waterflow" and that the pier was not reasonably accessible for inspection. He did not mention pier 3.

In December 1985, the senior construction specialist for the insurance carrier wrote in an internal memorandum that he believed that the Albany division conformed to the mandatory 2-year inspection cycle better than the other three divisions. However, he also believed that NYSTA bridge inspections were inadequate both in frequency and in detail and that management oversight was lacking in the review and control of bridge inspections. As a result, the insurance carrier's construction specialist recommended that his company not insure NYSTA bridges. After receiving responses from NYSTA on January 2, 1986, and examining the answers, the specialist changed his recommendation and advised coverage. Subsequently, the carrier insured the Schoharie Creek Bridge and some other bridges in the NYSTA system.

#### Qualifications and Training of NYSTA Bridge Inspectors

Three of the four assistant division engineers (bridges) were licensed professional engineers, and therefore qualified to be team leaders in bridge inspections. The assistant division engineer (bridges) in Albany, who was responsible for the inspection of the Schoharie Creek Bridge, was not a licensed professional engineer. However, his years of experience and his training qualified him to lead a bridge inspection team according to the NBIS. He had worked for the NYSTA since 1953, and before that, for the New York State DPW for 3 years.

In 1970, the assistant division engineer (bridges) in Albany attended the Bridge Inspection Manual 70 training course sponsored by the FHWA. He also attended a bridge inspection training program sponsored by the NYSTA in 1971, a bridge inspector's training course sponsored by NYSDOT in 1975, a course in investigation of structural failures given by the American Society of Civil Engineers in 1976, and a bridge maintenance and inspection seminar sponsored by the NYSTA and given by a consulting firm in 1982. This course was 2 days long and concentrated on inspection of the superstructure and the rating of various elements.

The last NYSTA annual bridge maintenance engineer's meeting before the collapse was held on January 10, 1986, at the NYSTA administrative headquarters in Albany. The day-long meeting covered the bridge maintenance program, bridge budgeting, and personnel attrition. The outline for bridge inspection included a discussion on underwater inspection and hands-on access.

The NYSDOT conducted a bridge inspector training course that NYSTA personnel sometimes attended. The last course given before the collapse was 5 days (3 full days and 2 half days) given at State facilities. The course included discussion on pier elements and stream channel elements. Field inspections were also made of two bridges. No NYSTA personnel attended the course.

### Equipment and Tools Used in Bridge Inspections

Each division bridge inspector had available a variety of tools and other equipment for use in bridge inspection. Specific inspection equipment included binoculars, boots and waders, probing rods (the Albany division had a 6-foot long straight rod), various hand tools, a 100-foot-long tape, and other measuring devices and equipment. The Albany division did not have a sounding line with weight for drop line readings. For access, exclusive of equipment required for the superstructure only, they had ladders, boats, and rigging. A 35-mm camera with accessories was also supplied. A NYSTA photographer was also available, if his services were desired.

### Guidelines for the Inspection of Substructures

Over the years, several documents were available to NYSTA and NYSDOT inspectors for their use. These documents were published by the AASHTO, FHWA, NYSTA, and NYSDOT. The documents required that inspections of all bridges over water include inspection of the substructure and inspection around the footings for evidence of scour. If erosion or scour was found, additional documentation was required.

AASHTO.--As previously mentioned in this report, the NBIS requires that bridge inspections be conducted according to the "Manual for Maintenance Inspection of Bridges" published in 1970 by the AASHTO (the "AASHTO Manual"). This manual recommends that a channel profile record should be maintained so that any tendency toward scour, channel shifting, degradation, or aggradation will be noticed. The manual states, "A study of these characteristics can help predict when protection of pier and abutment footings may be required."

The AASHTO Manual also recommends that footings be investigated for evidence of significant scour, using probing or diving as necessary. A 5-year inspection interval for these items is recommended except under unusual conditions. It states, "Particular attention should be given to foundations on spread footings where scour or erosion can be much more critical than a foundation on piles."

Another AASHTO guide, "The Manual for Bridge Maintenance, 1976" stipulates that scour is a complex problem and recommends that a geologist, hydraulic engineer, and structural engineer be consulted before correcting serious maintenance problems. This manual notes that erosion is a time-dependent process, but that the effects are particularly evident after rare and unusually severe floods. The manual indicates that to prevent or limit scour, the engineer should make a scour analysis at the site to assess the situation prior to undertaking any corrective action. Possibly the damage was caused by a flood well in excess of the design event. In this case, the only remedial action justified may be to return the structure to its original configuration, provided no significant channel modifications have occurred or are anticipated. The placement of heavy stone at points of potential scour may arrest minor scour conditions. In more serious cases, the manual recommends that sheet piling should be driven to a depth where rock or nonerodible soil conditions exist.

The manual recognizes that the difficulty in determining the turbulence through a bridge opening hinders the determination of how large and heavy riprap should be. However, the manual does provide a guide for selecting stone. The following table from the AASHTO manual suggests sizes. (The average velocity is to be calculated by dividing the discharge by the waterway area during the design flood).

<u>Average velocity</u>	<u>Average stone size</u>
up to 7 feet per second	6 inches [about 20 pounds]
7 -10 feet per second	100 pounds
10 - 15 feet per second	600 pounds

NYSTA.--In 1971, when the Authority hired the consulting firm of Byrd, Tallamy, MacDonald, and Lewis (BTM&L) to inspect its bridges, the consultants issued a report entitled, "Bridge Inspection Study and Development of a Long-Range Financing Program for Bridge Maintenance and Rehabilitation of New York State Thruway Authority." The report contained an appendix entitled "Bridge Inspection Manual for New York State Thruway Authority," which BTM&L developed for NYSTA. In this inspection manual, item 5-I states that the substructure should be checked and asks the question, "Is there any scouring or undermining of the foundations by improper drainage or stream channel flow?" Item 6-D states that the channel and channel protection should be inspected, and asks the question, "Is the channel protection or erosion control system (Gabions 53/ or riprap) adequate and in good condition?"

53/ Gabions--a basket or cage, usually of wire, containing stone and used to provide greater mass and hence, resistance to movement.

NYSDOT.--The NYSDOT published the "Highway Maintenance Guidelines" in 1972; this recommends that in-depth inspections should be made on all pier structures susceptible to scour. The guidelines recommend repairs with heavy stone fill or riprap "before scour progresses to a depth which might endanger the stability of a structure (1/2 of the thickness of pier footing)." These guidelines are still in effect.

In addition to the "Highway Maintenance Guidelines," the NYSDOT published its Bridge Inspection Manual sometime in the 1970s. It states that "occasionally, when a pier has scoured...look for undermining of stream piers. When water covers the footing which is shallow, probing with a stick or rod can be performed to determine the extent of scour." The manual does not explain the data that must be collected or the methodology to be used in the data collection.

A supplement to this manual entitled "Scour Documentation and Inspection Guidelines for Bridges over Water" was published in 1978 to describe the needed documentation. The supplement notes that Federal rules covering bridge inspection require scour documentation and foundation inspection for bridges over streams and waterways. This supplement subdivides bridges into four categories based primarily on the method of obtaining documentation and inspecting the bridge foundations.

The original NYSDOT Bridge Inspection Manual was superseded by the NYSDOT-BIM-82, which expanded the original manual to include a section on channel and scour documentation. The NYSDOT-BIM-82 requires inspections to include a channel profile for the length of the unit when water depths exceed 1 1/2 feet at the edge of the substructure unit. Additional requirements are recommended where scour is found.

### Medical and Pathological Information

All 10 vehicle occupants were fatally injured. Autopsies revealed that four fatally injured persons died from multiple fractures and internal injuries and two others died from drowning. Autopsies on the other recovered victims revealed that one died from traumatic head injuries, another from massive chest and abdominal injuries, and one from a combination of cardiorespiratory collapse, a laceration of the spinal chord, and fractures of the cervical vertebra.

### Survival Aspects

Virtually no survivable space was left in any of the accident vehicles. Upon impact, the roof on each car was crushed to the seat and the fiberglass cab of the truck tractor was shattered into several small pieces.

Seven of the ten fatally injured persons remained inside the accident vehicles. Seatbelts worn by the trapped victims had to be cut by rescue workers to permit the removal of the bodies. The bodies of the remaining three fatally injured persons were not found in their vehicles. Two of the bodies were recovered and one is still missing. One of the two recovered bodies was found downstream in the Hudson River about 45 miles from the bridge site.

### Emergency Preparedness

State.--The New York State Disaster Preparedness Plan, established in 1979, provides for preventive or mitigative actions before a disaster occurs, response actions when one occurs, and recovery action after it occurs.

Local levels of government (village, town, or city) are responsible for dealing with a disaster initially. (The next highest level of government, county, coordinates the local and county government response.) If their resources and capabilities are exceeded while responding to the disaster, the county can request State assistance through the State Emergency Management Office (SEMO). SEMO, which acts as staff to the New York State Disaster Preparedness Commission has a full-time staff to maintain and operate the State Emergency Operating Centers, which coordinate activities in their respective areas.

NYSTA.--The NYSTA is not an official member of the New York State Disaster Preparedness Commission. In fact, no State authority is a member of the Commission. However, the NYSTA does participate in meetings called by SEMO. The NYSTA and SEMO exchange information, and the NYSTA has participated in disaster drills.

County.--Montgomery County maintains an Office of Emergency Management (OEM) with a full-time Director and several support personnel. The Director of the OEM receives information (such as weather reports) from SEMO and coordinates rescue and other emergency resources when a potential disaster is identified within the County.

On April 3, the Montgomery County Director of the OEM received a call from the SEMO Regional Director and was informed that many areas of New York State could receive 5 or more inches of rain from April 3 to April 6, which could cause serious flooding. In addition, she was informed that the Gilboa Dam (approximately 58 miles upstream from the bridge site) was spilling over more than 1 foot.

Upon learning of the potential for flooding, the Montgomery County Director of OEM activated the County disaster plan. She notified the supervisors of each of the 10 towns and the five supervisors of the city of Amsterdam, within Montgomery County, of the potential for flooding and advised them to have their

salvaged. Testing included soil borings and bearing tests, concrete strength tests, span stiffness tests, erodability tests of glacier till, and a finite element analysis of the pier 3 substructure. In addition to this research, a hydraulics study, including computer mathematical modeling, and physical modeling, was done to further examine the factors that may have influenced the bridge collapse. The results of these tests and on-scene examinations are summarized below.

Examination of Wreckage.--Fifteen days after the collapse, a diver evaluated conditions at the upstream ends of piers 3 and 4. The diver found a 3-foot-high vertical soil ledge south of pier 3, and a tree and voids under the front of the pier footing.

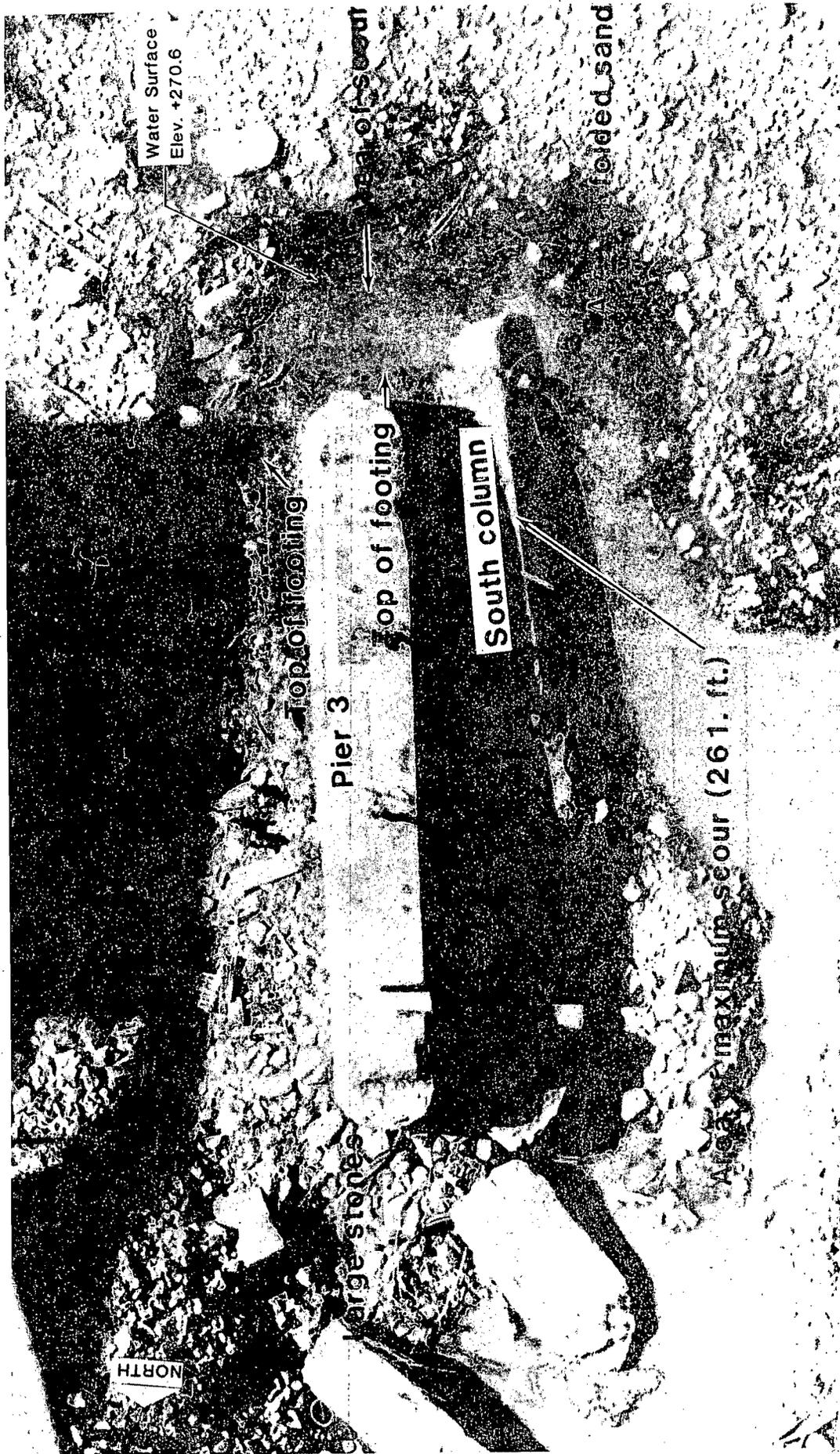
After a cofferdam was built around piers 2 and 3, the area was dewatered. A 10- to 14-inch-diameter hackberry tree was found trapped under pier 3. This tree was analyzed by staff at the State University of New York College of Environmental Science and Forestry. They determined that the tree probably fell into the water, in a dormant configuration, after the end of the 1986 growing season, indicating that it had been lodged there recently. The hackberry tree crossed and was bearing on a second tree of similar size and type. The second tree rebounded when cut a foot above the point where the two trees crossed.

The WJE investigating engineers believed that the scour conditions at the upstream end of pier 3 had not changed significantly after the collapse due to the damming of the creek, which occurred when the main girders of spans 3 and 4 fell, restricting the water flow around pier 3. WJE engineers believe that a significant amount of water was diverted towards the opening between pier 2 and the west bank. The top of the bridge debris functioned like a weir, causing water to pass over it and protecting the scour pattern.

In addition, the upstream half of the pier 3 plinth and footing rested within the scour hole, also protecting the streambed from further erosion. The WJE report also stated that the intact deck of span 4 may have deflected the creek flow away from the east side of pier 3, towards the opening below span 4 west of pier 4, and over the span 4 girders in the creek channel.

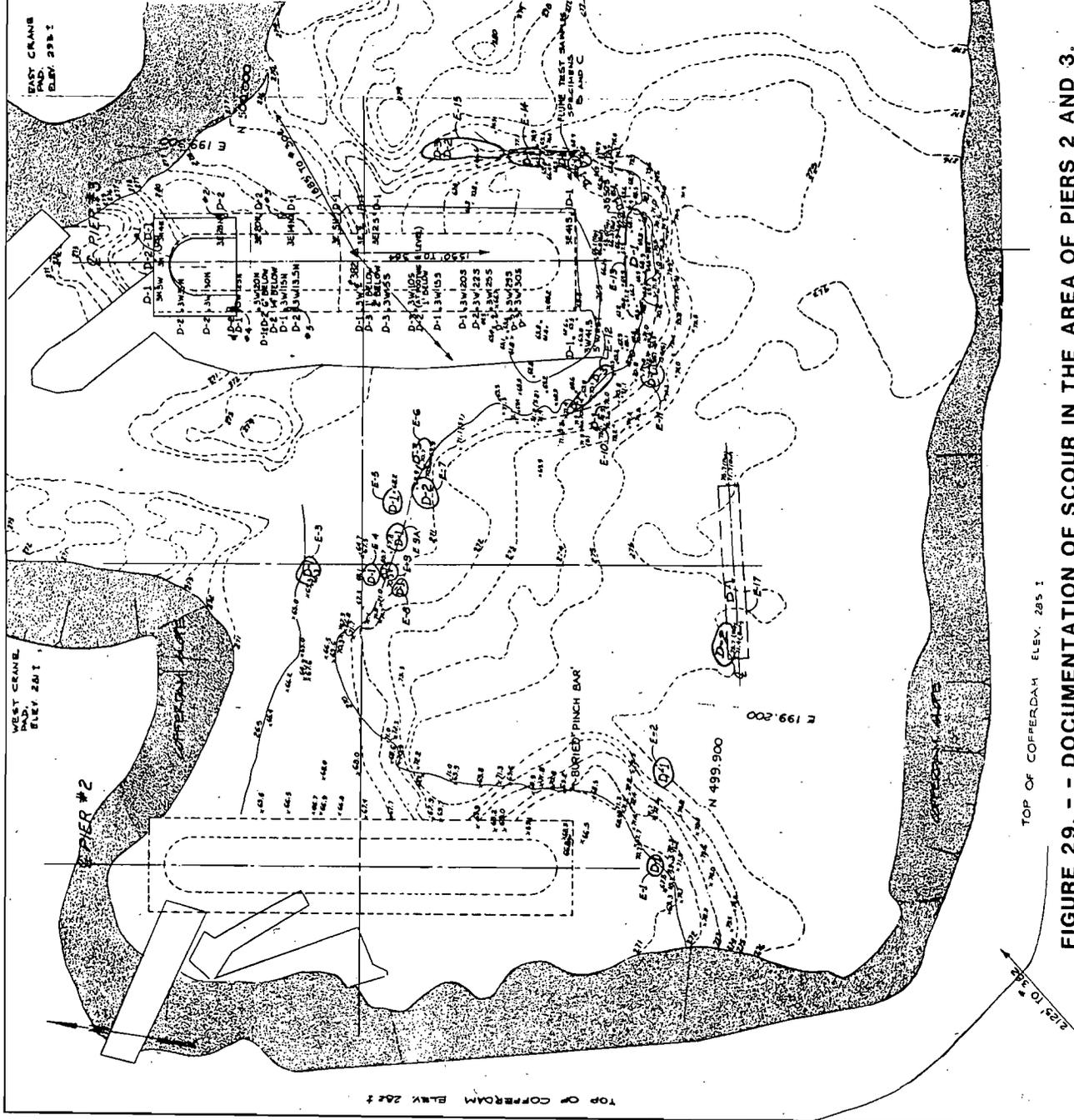
From a point 30 feet upstream of piers 2 and 3, WJE engineers observed that the creekbed sloped from an elevation of 275 feet to a depression in front of each pier. The creekbed elevation dropped abruptly at the depression, with its bottom elevation at about 265 feet. The entire depression was covered with a layer of cobbles. (See figures 28 and 29.)

A high pressure water pump was used to clean the scour hole surfaces of loose sand and silt. In some areas, the water jet had to be operated with special care to prevent eroding the sandier till-like soil and the stratified drift.



From WJE.

Figure 28.--Photograph of pier 3 after dewatering,  
before excavation of sides.



Survey control point established by C.T. Malle Associates, P.C.  
 Location of Structure D scouring surveyed. D - Location of top of conserve space. (S) - High-lighter special feature.  
 Location and number of plate bearing test.  
 Exposure number reviewed by geologist. Basic soil type of Structure D shown.  
 Location of soil sample taken by geologist. Location of geotechnical gradation test data presentation plates.  
 Elevation contours above Elev. +277 are taken from "RIS THROUGH (1-30) OVER SCHWARZ CREEK", New York State Department of Transportation, Mapping Service Bureau, Sheet No. DA, 6-11-97.  
 Scour hole contours shown are inter-polations of surveyed elevations taken by C.T. Malle Associates, P.C. under the direction of MCEI.

**SCHOHARIE CREEK BRIDGE INVESTIGATION**  
 MONTGOMERY COUNTY NEW YORK  
 NEW YORK STATE THRUWAY AUTHORITY  
 200 SOUTHERN BOULEVARD ALBANY NEW YORK  
**MUESER RUTLEDGE CONSULTING ENGINEERS**  
 708 THIRD AVENUE, NEW YORK, N.Y. 10017  
 SCALE: 1"=10'-0" DATE: 8/97  
 DRAWN BY: PS DATE: 7/14/97  
 CHECKED BY: PS DATE: 6/18  
 DESIGNED BY: PS  
 CONDITIONS WITHIN DEWATERED COPPERDAW S-1

TOP OF COPPERDAW ELEV. 285.1  
**FIGURE 29. - - DOCUMENTATION OF SCOUR IN THE AREA OF PIERS 2 AND 3.**  
 1575 TO 304  
 10 3/2

A backhoe was used to expose the footing of pier 3 to a depth 1 foot below the footing at the north end and to the scour hole surface around the upstream (south) face. All scour surfaces were cleaned by hand. During this excavation, no riprap was found below an elevation of 275 feet, which was the top of the footing. However, hammers, wrenches, and other miscellaneous debris were found at elevations below 275 feet. Soil samples were taken directly beneath the pier and around its perimeter. These samples were classified into three categories of stratum: till-like silt, stratified drift, and out wash. The soil material excavated below the elevations where riprap was present was random backfill consisting of sand and gravel.

Examination for Scour.--After the bridge wreckage was removed and the loose sand and silt was excavated from around pier 3, the scour patterns were revealed. On the east side, the exposed scour pattern extended northward about 40 feet from the edge of the footing and about 34 feet on the west side. Measurements taken in the scour hole indicated that the maximum depth of scour was about 9 feet below the footing, just west of the footing of pier 3. Backfill and gravel were found on the east side of pier 3.

The bridge wreckage was removed and loose sand silt was excavated from around pier 2. The exposed soil pattern revealed that scour had extended as much as 40 feet southward from the upstream edge of the footing. The maximum depth of scour beneath the footing of pier 2 was about 5 feet. Toward the rear of the pier, the glacial till appeared to be eroded into a ramp.

Riprap.--WJE engineers also inspected the streambed around pier 3 to determine if riprap was present at the bridge site. A few large pieces of stone were found at the southeast corner of the footing and a moderate amount of river alluvium had been deposited in the scour hole on the east side of pier 3 below span 4. However, the west side of pier 3, between the footing and the fallen south column, was free of alluvium for the full depth of the footing where the column pressed into a mound of erosion-resistant, fine-grained soil. A total of 151 large rocks were found in the excavation at pier 3. A few other large rocks were found immediately adjacent to the pier at the southeast corner of pier 3, but most were concentrated at the northeast corner of the pier. Angular rocks were found around the sides of the pier and strewn over the creek bed to the west. Rounded rocks common to the creek bed, but noticeably larger, were found downstream of the pier.

Approximately 34 of the larger rocks from around pier 3 collected over the course of the work were stockpiled in the west storage area. These rocks were categorized by shape as round (18), semi-round (5), or block (11). The average weight of the round and semi-round rocks was calculated to be 1,550 pounds, with a median weight of 1,000 pounds. The block shaped rock had an average weight of about 830 pounds and a median weight of 800 pounds. Two of the round rocks were calculated to weigh 5,000 to 7,000 pounds.

Soil Borings and Bearing Tests.--MRCE took soil borings and conducted soil bearing tests to determine if the subsoil base on which the spread footings were constructed met the designer's minimum specified load bearing requirements. Of the 25 borings taken, 18 extended to bedrock. Numerous soil tube samples were taken below the piers and approach embankments. Borings 2.5 to 3.5 inches in diameter were made to obtain undisturbed tube samples. In some holes, a piezometer 55/ was installed to monitor ground water level. Soil samples were analyzed for type, water content, liquid limit, plasticity index, grain size distribution, compression strength, confining pressure, strain at failure, initial wet density, dry density, and elastic modulus. 56/

In general, MRCE reported that the subsoil base supporting the spread footings consisted of bedrock with shale and limestone found at elevations of 225 to 230 feet. The bedrock was overlain by 25 to 30 feet of very compacted glacial till, which was covered by a second layer of alluvial deposits of various depths. Generally, these borings and the borings taken at the same site in 1951 did not differ significantly except in the area south of pier 3. Soil borings in that area indicated vertical bedding planes with sand material that had been folded. Based on the recent boring tests, the soil under the pier footings was calculated to have had an ultimate bearing capacity of a minimum of 13 tons per square foot (tsf) each. The original design was based on a bearing capacity of 3 tsf.

Soil Erodability Study.--Cornell University conducted a soil erodability study to determine the speed at which significant erosion would occur when subsoil samples were subject to different velocity stream flows. Three samples of till-like stratum were hand-carved, placed in rigid 1-cubic-foot wood boxes, and taken to the Hydraulics Laboratory of Cornell University. There, the samples were trimmed to 7-by-12 inches and placed in a flume 16 feet long and 8 inches wide. The maximum velocity that could be attained in the flume was 10 feet per second.

Erosion of the soil surface was measured by computing the soil volume lost at the exposed surface. An obstruction was placed in contact with one sample to determine if the local turbulence created would influence the erosion magnitudes and rates.

55/ Piezometer - a device that measures the static pressure of a fluid in motion without disturbing the velocity during measurement.

56/ An expression of the maximum stress forces for which the material acts elastically.

The results are summarized below:

Sample A represented dense, broadly graded, till-like stratum from the Mill Point Bridge vicinity. The texture of the eroded surface was rough, with small to medium-sized gravel contained within a matrix of fine-grained, slightly clayey silt. Loss of soil at velocities up to 4.4 fps was low, but the rate of loss gradually increased with time. At 10 fps, erosion increased markedly, but the rate slowed with time.

Sample B represented the sandier of the till-like materials of stratum found under pier 3. At flows up to 6.25 fps, erosion was minimum. Erosion increased drastically and accelerated with time at a velocity of 8 fps.

Sample C was comprised of mostly fine-grained silt or slightly clayey till-like material of stratum under pier 3, and had more apparent stratification. The total loss of soil was only about half that of the other two samples, up to 7.5 fps. At 8 fps, the amount of soil erosion increased significantly.

While sample C eroded less than the other two samples, the erosion that did occur was magnified by the presence of the obstruction. Most of the erosion that occurred on the surface of sample C was immediately upstream of the obstruction.

MRCE concluded that these test results, while difficult to reduce to quantitative terms, did indicate that till-like material will erode significantly at a velocity of 8 to 10 fps. Those materials that exhibited a slight plasticity, such as samples A and C, continued to erode but at a slower rate after the flow velocity exceeded 8 fps. On the other hand, the sandier material of sample B eroded at an increasing rate when the velocity exceeded 8 fps.

In general, all of these dense soils were relatively stable at low velocities. The more plastic soils tended to form a somewhat resistant surface, and the sandier material was the most vulnerable to scour. Thus, the texture of the soil surface influenced both the magnitude and rate of surface erosion.

Concrete Strength Test.--WJE conducted concrete strength tests to determine if the concrete used in the bridge structural members met the minimum strength requirements specified in the bridge design plans. A total of 40 compressive strength tests and 8 split cylinder tensile strength tests were conducted. Samples were obtained from the footings, plinths, and plinth reinforcements of each pier.

The tests were conducted on 4-inch-diameter cores, except one test, which was conducted on a 5-inch diameter core. The cores were tested in accordance with the American Society for Testing and Materials (ASTM) C42 "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete Specimens" and ASTM C496 "Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens."

The minimum measured tensile value was 550 psi. The tests for all samples except the plinth reinforcement indicated an average [ultimate] compressive strength of 6,020 psi with minimum compressive strength value of 4,160 psi. Tests for pier 3 averaged 6,200 psi dry and 6,040 psi wet. The plinth reinforcement concrete cast in 1957 had an average (ultimate) compressive strength of 8,790 psi. Samples from the east abutment indicated an average [ultimate] compressive strength of 6,200 psi. General notes on the original drawings for the Schoharie Creek Bridge called for a minimum ultimate compressive strength of the concrete of 3,000 psi at 28 days. The maximum tension loading allowed by the AASHTO design manual for footings of plain concrete with a minimum ultimate compressive strength of 3,000 psi was 90 psi.

Finite Element Analysis of Pier.--The evaluation of data obtained during the field investigation indicated to WJE that the collapse was caused by movement of pier 3. For this reason, WJE undertook a detailed finite element analysis (FEA) of pier 3 to determine the internal stresses exerted on the plinth with various amounts of undermining beneath the footing. The FEA also determined if these stresses may have caused the major rupture that was observed in the plinth after the collapse.

To consider a variety of loading conditions, WJE developed a model of pier 3 using three-dimensional finite elements. The pier consisted of 1,472 solid 8-node brick-type elements, connected to mesh of 2,712 nodes. WJE assigned a value of 500 kips per cubic foot 57/ for soil stiffness based on an analysis of all soils tests.

For each loading condition analyzed, corresponding finite elements were modified accordingly. In all loading conditions, a crack located approximately 7.5 feet north of the center of the plinth was built into the model to simulate the structure conditions before the collapse.

Using the finite element model, WJE calculated the stresses in each element of the pier for a variety of loading conditions. These stresses were then compared with the ultimate tensile strength of concrete with a reinforced cap to estimate the strength at which the plinth would break.

57/ Kips--1,000 pounds per square inch.

The WJE analysis concluded that the forces in both columns and piers were within acceptable limits for gravity loading conditions. Wind and water current loads were low and, when they were included, did not appreciably increase the stresses in the piers. However, scour under the upstream footing of pier 3 greatly increased stresses in the plinth with the reinforced cap, although at first only in the central portion of the plinth, which was reinforced with the cap. For large amounts of scour beneath the footing, the flexural stresses on the downstream end of the plinth increased rapidly. When scouring reached 25 to 30 feet longitudinally in front of the pier and beneath the south end of the footing, flexural stress at the upper surface of the "lightly" reinforced section of the plinth (0.5-inch diameter bars spaced 18 inches apart) exceeded the tensile strength of the concrete. Under these conditions, a fracture began at the top of the plinth at an angle perpendicular to the flexural stresses, about 25 degrees from the vertical. The crack propagated until it reached the preexisting vertical crack near the center of the plinth. At that point, the upstream portion of the plinth separated from the downstream portion and underwent a large amount of rotation and downward movement.

WJE stated that the initial design without the plinth reinforcement cap carried a considerable bending moment between the two columns. Maximum tensile flexural stress, extrapolated from the stress at the center of the element, was about 125 psi at the top. This stress in combination with other factors may have caused the vertical crack observed in 1955.

Hydraulic Study.--The Safety Board contracted with Resource Consultants Inc. (RCI) and Colorado State University (CSU) in Fort Collins, Colorado, to study the role of scour in the collapse. This study was co-sponsored by the NYSTA. RCI and CSU conducted on-site examinations, collected hydraulic data, developed flood hydrographs, performed a hydraulic water-surface profile computer analysis (WSPRO), constructed two- and three-dimensional physical models, and evaluated riprap stability based on the combined analyses. The study included a comparison of the characteristics of the 1955 and the 1987 floods, because the 1955 flood had both greater peak flow and volume, and yet the bridge survived the flood with little noted damage.

High water marks were needed to calibrate the computer water-surface profile model. RCI and CSU based these marks on high water marks surveyed by the USGS along the banks of the Schoharie Creek and the Mohawk River. They also used Burtonsville gage data, survey information from MRCE, and photographs taken at various times, including shortly before and after the failure. High water marks just upstream of the bridge measured after its collapse had to be adjusted because the presence of the bridge debris raised the water level at that location. The water surface elevation at the bridge site was estimated from photographs taken of the collapse from the Fort Hunter bridge. The adjusted high water level at the bridge was calculated to be 296 feet.

The Army Corps of Engineers' Hydraulic Engineering Center (HEC-1) Flood Hydrograph Model was used to simulate the water in flow runoff between Burtonsville and the bridge. Results of the HEC-1 model calculations indicated that the peak flow of the April 1987 flood was attenuated from 64,900 cfs at Burtonsville to 63,100 cfs just upstream of the bridge. The model indicated that for the 1955 flood, the water inflow from Burtonsville to the bridge was greater than the runoff, increasing the flow just upstream of the bridge to 76,600 cfs from 76,500 cfs at Burtonsville.

West of the Schoharie Creek Bridge was a cornfield and an opening for the bridge at Fort Hunter. During both floods, water passed over the field and under the Fort Hunter bridge. Based on pictures and high water marks, RCI estimated that in 1955 and 1987, 3,000 and 1,000 cfs, respectively, were diverted under this bridge instead of under the Schoharie Creek Bridge. Thus, the peak flows under the Schoharie Creek Bridge in 1955 and 1987 were estimated to be 73,600 and 62,100 cfs, respectively. According to NYSTA, there was erosion under the Fort Hunter bridge during the 1955 flood. This may have slightly increased the peak discharge under that bridge. If this occurred, the peak flow estimated at the Schoharie Creek Bridge would have been slightly lower.

Computer Modeling.--RCI used the WSPRO one-dimensional flow computer model, developed by the USGS under funding from the FHWA, to quantify flow characteristics, such as water surface elevations and flow velocities at given discharges. The WSPRO model was chosen because it provides a detailed hydraulic analysis of flow through bridge openings. Input data required for the model included creek bed channel cross section data, roughness coefficients, channel and flood plain data, and bridge geometry. It also was necessary to determine the water level of the Mohawk River at the mouth of the Schoharie Creek to establish downstream boundary conditions. A 2-mile segment beginning 4,500 feet upstream of the bridge site and extending to the Mohawk River was selected for modeling. The model was initially calibrated using the 1987 flood data. Initial estimates of the input data were chosen, computer runs were made, and certain water elevation levels were compared with known high water marks. The roughness coefficient was adjusted slightly until the calculated water elevations compared favorably with the known high water levels. The model was used to calculate velocities and water elevations for the 1955 flood without adjustments for changes in roughness, channel cross section, and other factors.

For the 1987 flood, the average horizontal velocity in the approach section to the bridge was calculated to be 10.3 fps. The velocity of the flow over the cornfield to the southwest was calculated to be 3 to 4 fps. The average velocity of the creek at the bridge was calculated to be 11.4 fps. The calculated water surface elevation immediately upstream of the bridge was 294.7 feet.

During 1955, the average velocity in the approach section of the stream was calculated by the computer model to be 9.8 fps. At the bridge, the average velocity was calculated to be 12.4 fps. The computer model indicated that, for the 1955 flood, backwater from the bridge increased the upstream water surface elevation less than 0.5 feet and increased the velocity by 2.0 fps.

The RCI study indicated that although the 1955 discharge passing under the Schoharie Creek Bridge was about 19 percent larger than the 1987 discharge, the 1955 mean velocity through the bridge was not significantly greater (12.4 in 1955 versus 11.4 fps in 1987). According to RCI, the lack of a significant difference in velocity was a direct result of higher tailwater elevation (296 versus 294 feet) in the Schoharie Creek during 1955. Therefore, RCI concluded that even though the 1955 flood had a larger peak flow, the erosion potential under peak flow conditions was only slightly larger than for the 1987 flood. 58/

Velocities were calculated at some locations across the channel using a stream-tube modification 59/ of the one-dimensional WSPRO computer model. However, this was limited by the input data necessary for this analysis and the unique physical characteristics of the bridge site. The large bend upstream of the bridge and the flow from the cornfield reentering the channel immediately upstream of the bridge created a complex flow pattern approaching the bridge, limiting the ability of even the modified WSPRO model to produce a sufficiently detailed velocity distribution across the channel. Therefore, physical model studies were performed to obtain a more detailed picture of the velocity variations than the WSPRO model provided.

Physical Models.--Using the Hydraulic Laboratory at its Engineering Research Center, CSU built a 1:50-scale, three-dimensional model of the Schoharie Creek and flood plain area in the vicinity of the bridge in its 20- by 100-foot recirculating river flume. The model represented 2,000 feet of creek channel upstream and 600 feet downstream of the bridge. The model of the creek reach had a cover of mortar to fix the slope, but the creek channel was modeled in compacted sand to allow scour.

58/ The backwater from the Mohawk River did not significantly affect the velocities in the Schoharie Creek at the bridge.

59/ The division of the stream into small vertical sections that are combined for a cross-section.

In the three-dimensional model, tests were run to simulate the conditions of the flood as shown in the WSPRO results (i.e., a tailwater of 294.0 and a flow of 62,100 cfs). Velocity measurements were made across the stream using a Marsh-McBurney magnetic current meter. <sup>60/</sup> CSU also modeled the 1955 flood and 10 other runs were made to analyze other conditions and to assess the sensitivity of the model to slight changes in tailwater and flow. The runs CSU thought best modeled the 1987 and 1955 floods are shown in figure 30. A test matrix and most of the other model runs are included in appendix F.

The velocity distribution across the creek at the bridge for the flood of 1987 (with a tailwater elevation of 294 feet) is shown in figure 30. The greatest velocity, 12.5 fps, occurred in the channel between piers 3 and 4. Directly upstream of pier 3, the velocity was about 10.8 to 11.2 fps. The surface flow upstream of pier 3 was aligned with the pier. However, the bottom flow approached the pier from the east about 5 degrees out of alignment. At pier 2, the velocity was 7.5 fps. The velocity at pier 2 was aligned with the pier on both the surface and at the bottom, even though the flow coming from the cornfield had a dramatic appearance on the surface when re-entering the main flow.

The WSPRO model was used to determine average stream velocities of various flow discharge rates. However, as shown in the three-dimensional model, velocities varied across the stream. At pier 3, during flows of 60,000 cfs or more, the velocity was 1.1 times the average velocity predicted by WSPRO. However, with a flow of 30,000 cfs, pier 3 had a velocity 0.89 times the average velocity predicted by WSPRO. The reader should note this variance in the results of the computer model and the physical model. CSU has indicated that the accuracy of the velocities in its three-dimensional physical model is plus or minus 1.0 foot. Table 5 shows the various discharge rates, the WSPRO mean velocity, and the observed velocity at pier 3 for two conditions:

Table 5.--Peak discharge vs. WSPRO mean velocity at pier 3

Peak Discharge cfs	WSPRO Mean Velocity ft/sec	Velocity at pier 3 from the 3-dimensional model ft/sec
10,000	3.6	---
20,000	5.5	---
30,000	7.0	6.2
40,000	8.2	---
50,000	9.4	---
60,000	10.3	11.3

<sup>60/</sup> Magnetic current meter--a device that measures velocity without disturbing the flow.

Schoharie Creek Velocity Cross Section  
 Q = 62,100 cfs TW = 294' X-Sect U20

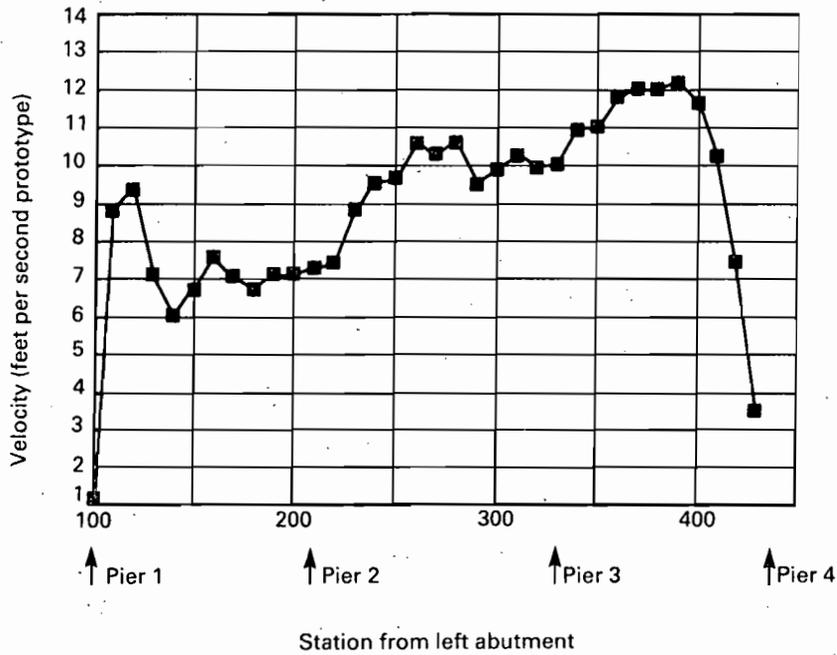
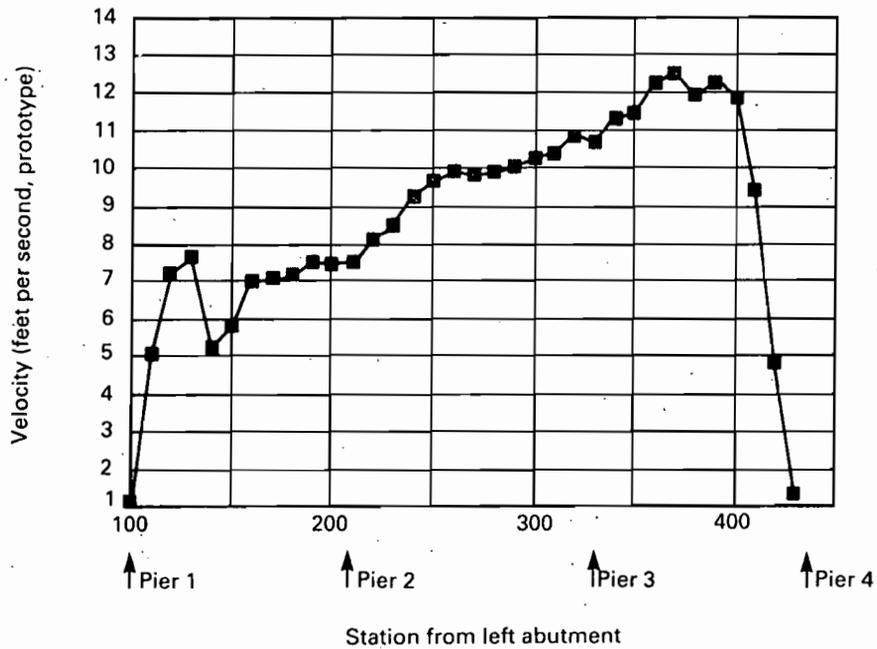


Figure 30.--Velocity profile upstream of the bridge for the 1987 flood condition.

Schoharie Creek Velocity Cross Section  
 Q = 73,600 cfs TW = 296' X-Sect U20



From CSU.

Figure 31.--Velocity profile upstream of the bridge for the 1955 flood condition.

The velocity distribution across the creek for the flood of 1955 (with a tailwater elevation of 296 feet) is shown in figure 31. Again, in this test run, the greatest velocity, this time 12.1 fps, occurred in the channel between piers 3 and 4. Directly upstream of pier 3, the velocity was 10.0 fps.

In the model, scour at pier 3 formed a distinctive "horseshoe vortex" around the front of the pier and trailed away from the pier on the west side while hugging closely to the pier on the east side (figures 32 - 36). All test conditions created a scour hole that undercut the south portion of the pier 3 footing, some by as much as 50 percent. Twelve runs were made with the model using various flow and tailwater conditions; in one run, debris was dropped into the stream after scour had developed sufficiently at pier 3 to simulate the collapse of piers 3 and 4. The results for this model are in appendix F.

The deepest point of scour observed around pier 3 was under the south edge of the footing, where an elevation of 260 feet was reached. The soil bed did not erode at the north end of pier 3, which was around the downstream edge of the spread footing, and in some of the runs, soil material was deposited in this area. The streambed itself scoured around the south half of pier 3 in a horseshoe pattern, sloping down from the streambed toward the pier. This slope formed a ramp on the sides of the pier and a ledge 5 to 10 feet south of the pier.

With the simulated bridge intact, scour at pier 2 never undercut the bottom of the footing, and under conditions of lower tailwaters, the bottom edge of the upstream face of the spread footing was not exposed. Once simulations of the debris from spans 3 and 4 were placed into the flow, pier 2 was undercut rapidly.

A large two-dimensional model of pier 3 representing a 120-foot-wide slice of the stream was built at a scale of 1:15 and was placed in an 8- by 200-foot variable slope recirculating flume. The flume was 4 feet deep. The model was built to observe the flow characteristics and variability in velocity at different locations around pier 3 under flow conditions representative of the 1987 flood (i.e., tailwater of 294 feet and flow of 62,100 cfs). To allow flow observation, the pier was modeled in clear acrylic up to an elevation of 310 feet (6 feet above the plinth reinforcement), where the model ended. The model was operated long enough for a scour hole to develop and then detailed horizontal velocity measurements of the horizontal velocity component were made at five locations around the pier at six different elevations. (See table 6.) Location A was 5.4 feet south of the footing on its extended centerline. Locations B and C were 5.5 feet east of the footing and 18.4 feet and 33.4 feet, respectively, north of the south end of the spread footing. Locations D and E were 5.5 feet west of the footing and 18.4 feet and 33.4 feet, respectively, north of the south end of the spread footing.

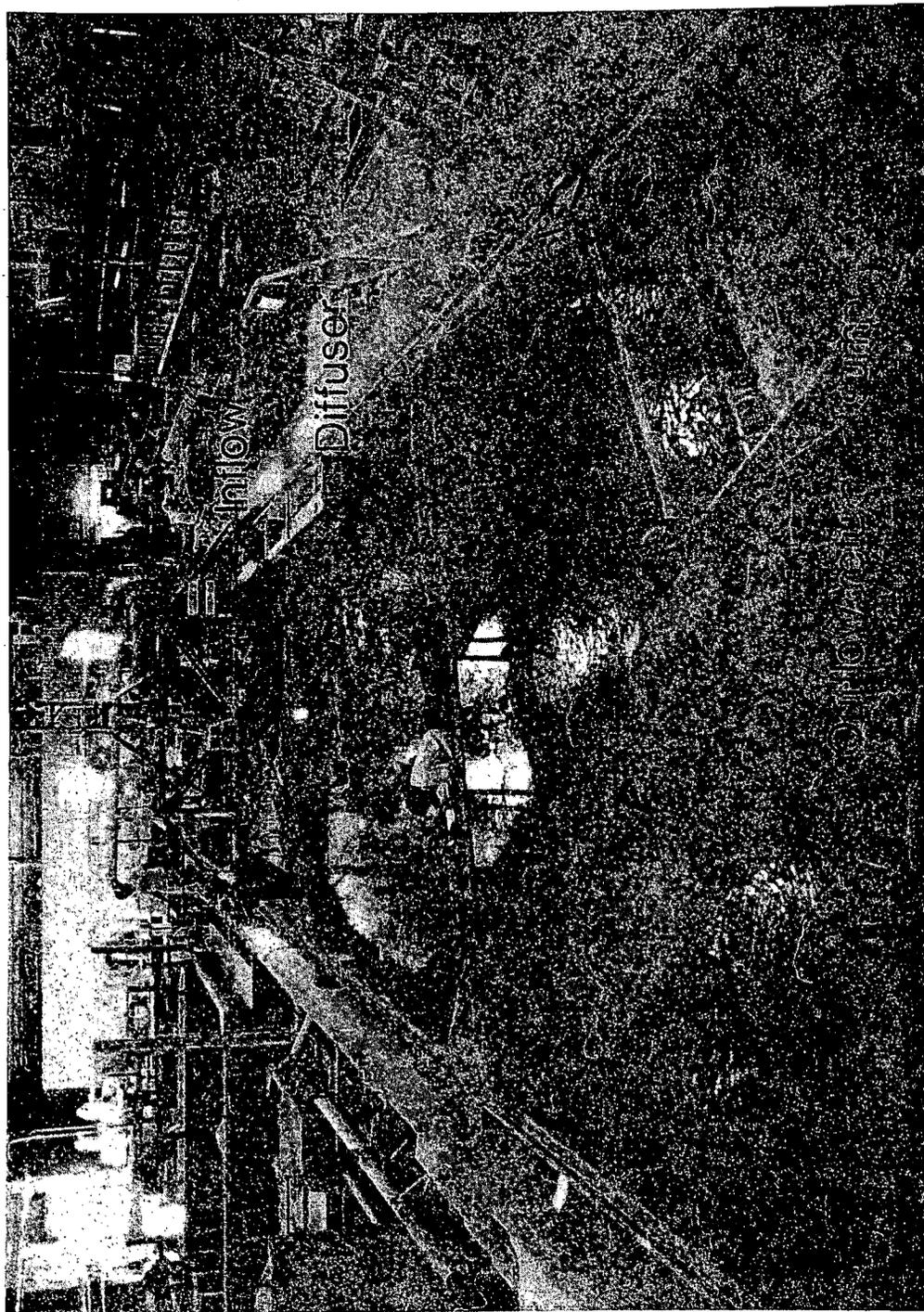


Figure 32.--Overall view of flume model (1:50).



Figure 33.--View of the 3-D model at pier 3 during the test run to duplicate the 1987 flood conditions. Note the scour hole pattern and the pattern of the surface water.

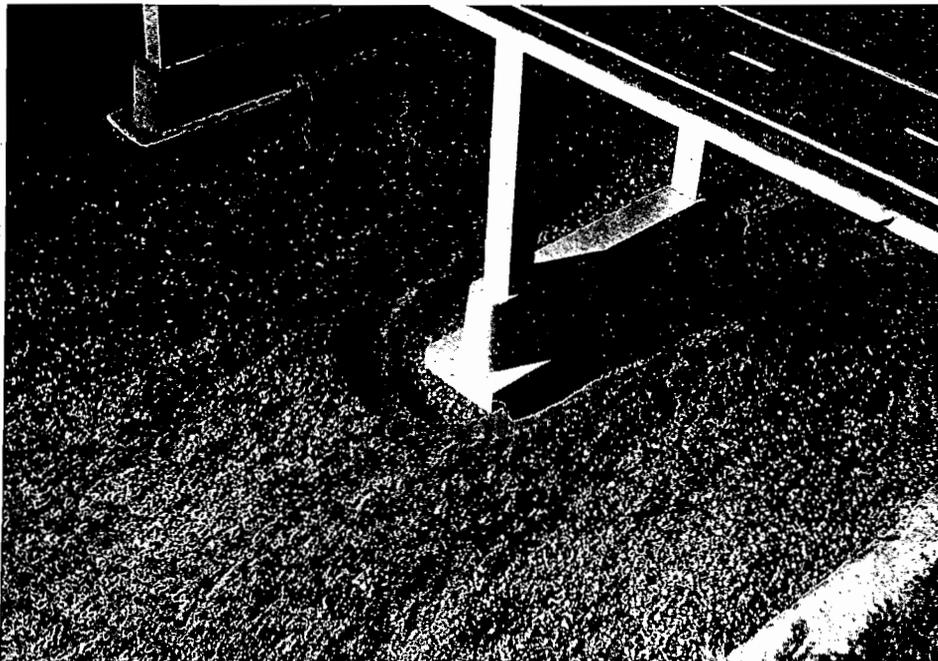


Figure 34.--Pier 3 from the right bank showing the scour hole in the dry channel after simulation of the 1987 flood prior to bridge failure. Pier 2 at the top of the picture has considerably less scour than pier 3.

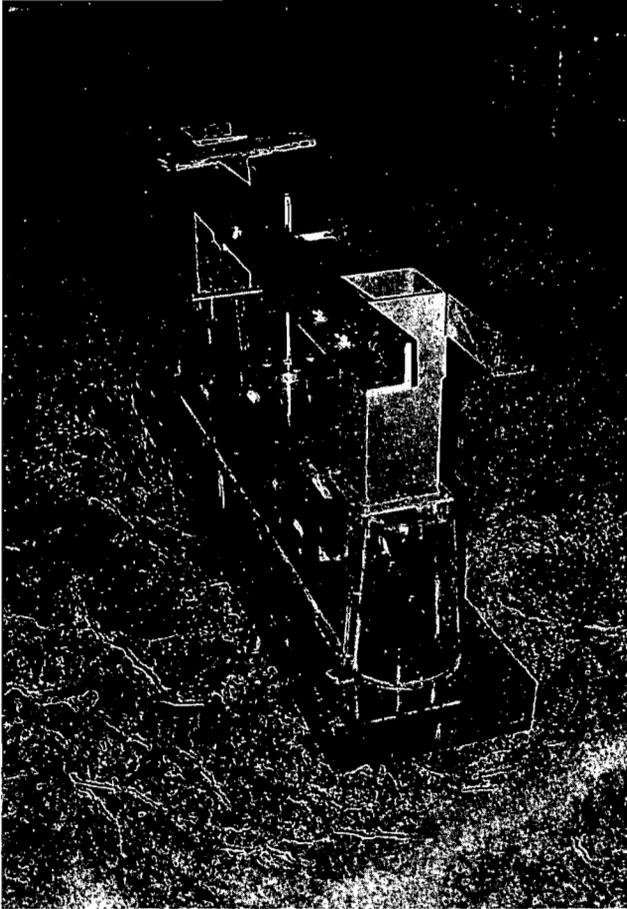


Figure 35.--View of the plastic 1:15 scale (2-D) model in the flume.

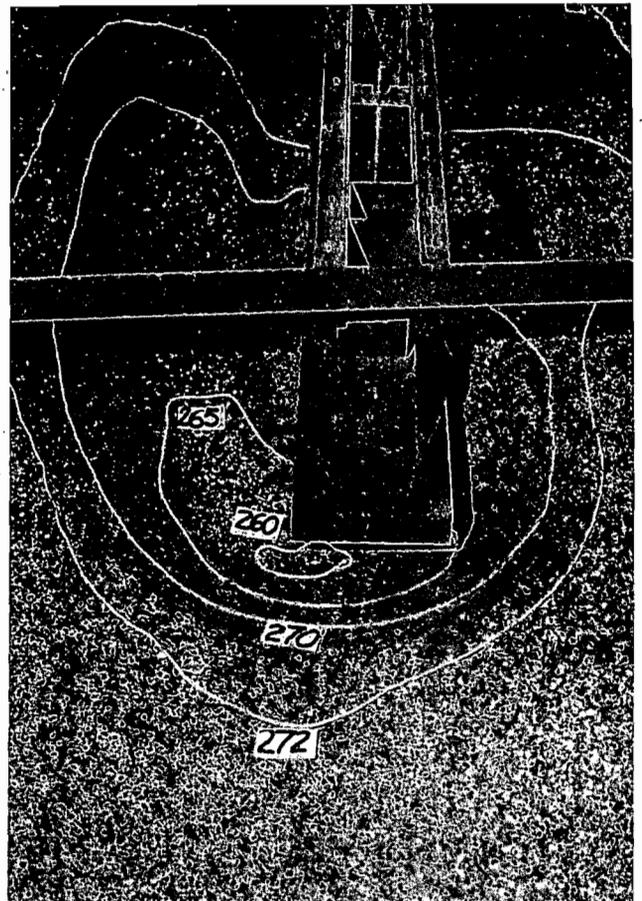


Figure 36.--Contour of the dry scour hole under pier 3 for the 1987 flood conditions in the 1:15 model.

Table 6.--Velocity (fps) measurements at various creek locations around pier 3

Position elevation (feet)	<u>Center</u>	<u>Right side</u>		<u>Left side</u>	
	A	B	C	D	E
265	4.2	6.6	5.8	7.4	-
270	6.2	8.5	7.4	8.9	-
275	6.8	9.3	8.7	7.9	8.1
280	8.9	10.0	10.3	10.4	8.5
285	9.5	10.8	10.9	11.2	10.3
290	10.8	12.2	11.1	11.6	9.3

An observer noted that bursts of turbulent flow were followed by periods of relative calm. When looking down the column during the test run, an observer saw the scour hole forming under the spread footing. During the periods of turbulent flow, strong vortices formed a roller 61/ in front of the pier, which churned material away from the front of the footing. During the periods of relative calm, material was carried under the footing and out the far side.

CSU also calculated the lateral forces that could be generated at the piers during the peak stream flow. They concluded that the forces were insignificant when compared with the massiveness of the pier structure in the water.

#### RCI-CSU Analysis of Scour

Total scour of a streambed generally is the sum of long term scour, contraction scour, and local scour. Long-term changes in a river bed include the aggradation (build-up) or degradation (reduction) of the streambed elevation by natural or man-made causes.

RCI and CSU aerial reconnaissance of Schoharie Creek revealed numerous exposed bedrock outcrops. Between Fort Hunter and Mill Point, there are two major outcrops--the first about 2.5 miles south of the Thruway and another about 4.0 miles south of the Thruway. At least two more outcrops cross the creek bed between Mill Point and the Burtonsville Bridge, and the USGS gage at Burtonsville is located in a reach of significant bedrock control. 62/ Because bedrock is not subject to erosion within 30 to 40 years, RCI and CSU concluded that the bottom elevation of the streambed of the Schoharie Creek remained unchanged in the area of the bridge.

61/ Roller - a phenomenon that occurs in front of the piers due to water turbulence.

62/ A control establishes elevation.

Using data from the time of the bridge construction and the time of its collapse, RCI/CSU compared the elevation of the streambed immediately upstream and downstream of the bridge. The comparison indicated that the general streambed configuration remained relatively unchanged after being subjected to numerous floods. In view of this, the RCI study concluded that the long-term change in streambed elevation at the Thruway bridge site was not a factor in this collapse.

Contraction scour results from a narrowing of the streambed channel and the consequent restriction in flow caused by either natural or man-made conditions, such as the construction of a bridge in a channel. Generally, when the flow is constricted, the velocity of the flow increases, increasing the potential for scour.

As the Schoharie Creek approached the bridge, water flowed over the banks and into a cornfield southwest of the bridge. The flow over the cornfield was forced back into the channel near the bridge by the build-up of fill that raised the approach to the bridge at the west abutment. This restriction to the creek's flow was initially thought to have the potential for contraction scour. CSU calculated that the bed of the stream could have been expected to scour 3.1 feet deep across the entire width of the channel. However, due to the concentration of the flow to the east bank, as a result of the bend upstream of the bridge, and due to the large cobbles on the streambed that resisted movement, contraction scour did not occur across the entire bed of the channel. Some contraction scour may have occurred, however, between piers 3 and 4 where the velocity of the stream was the highest and was capable of moving the large cobbles. In this area, 1 to 2 feet of alluvial deposits may have been removed down to the glacier till. After pier 3 collapsed and spans 3 and 4 fell into the stream blocking the flow, the flow was concentrated near pier 2. This area showed signs of contraction and local scour after the area was dewatered.

Generally, local scour is deeper than long-term or contraction scour, often by a factor of 10. However, when major changes in the stream conditions occur, such as the construction of a large dam upstream or downstream or the severe straightening of the stream, long-term scour causing bed elevation changes can be the larger element in total scour.

Local scour can occur in the vicinity of a pier or abutment when the velocity of the flow reaches a high enough level. Flow is accelerated around the obstruction and as the velocity increases, the flow becomes turbulent. This creates vortices, which can remove soil material adjacent to these flow obstructions. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth of scour increases, the

velocity and thus the strength of the vortex or vortices is reduced at the bed of the scour hole and the rate of transport of material from the hole is reduced. When equilibrium is reestablished, scouring ceases.

The vortex that fits around the base from the leading edge downstream on both sides of the pier is often shaped like a horseshoe. This horseshoe vortex is very strong and is the principal mechanism for removing the bed material around piers. Downstream of the pier, a vertical (wake) vortex is generated. This wake vortex removes some of the material at the downstream sides of the pier and deposits some material immediately at the rear of the pier. As a result, there is often a trough in the bed on both downstream sides of a pier and a ridge of deposited material directly behind the pier.

The RCI/CSU study concluded that several factors may affect local scour. For example, an increase in pier (or footing) width increases scour depth. At the Schoharie Creek Bridge, when the scour extended vertically below the 11-foot-wide plinth, exposing the 19-foot-wide spread footing, it increased the ultimate depth of the scour.

The depth of flow also affects the depth of scour. As the depth of the flow increases around a pier, the depth of scour increases. Also, because an obstruction in the water will increase the height (depth) of the water, the scour depth may also increase.

Velocity of the approach flow may influence the depth of scour. The greater the velocity, the deeper the scour. Water turbulence also affects scour depth; the more turbulent the water, the larger is the material that can be moved.

Length of a pier has no appreciable effect on scour depth as long as the pier is parallel with the flow. However, when the pier is at an angle to the flow, the length has a very large effect on scour. For any given angle of attack, doubling the length of the pier can increase the scour depth by as much as 33 percent. RCI/CSU stated that the 5-degree angle of attack at the bottom of the stream on pier 3 may have increased the depth of scour.

Similarly, according to the RCI/CSU study, the shape of a pier has a significant effect on scour. Streamlining the front end of a pier reduces the strength of the horseshoe vortex and thus the depth of the scour. The maximum scour of a square-nose pier will be 20 percent larger than that of a sharp-nose pier and 10 percent larger than that of a cylindrical or round-nose pier. (The Schoharie Creek Bridge plinth was rounded on the end and as scour proceeded, the footing, which was rectangular, was exposed.) Streamlining the downstream end of piers will similarly reduce the strength of the wake vortices.

Very large particles in the bed material, such as cobbles or boulders, may armor plate a scour hole. In the scouring process, the finer material and the larger particles will scour out of the hole; as the scouring proceeds, some larger particles move lower in the hole. The water velocity at the bottom of a scour hole decreases until a depth is reached at which the velocity in the hole is too slow to move the large particles. These particles form a layer that protects or armors the hole from further erosion. This armor plate can be broken by flows with greater velocities. Even riprap placed on the bed around a pier or abutment or partially buried in the bed can be eroded by the vortices generated by turbulent flood waters if the riprap is not properly sized and placed.

Based on the observations in the two-dimensional physical model, the RCI/CSU study concluded that flow around pier 3 at the time of the 1987 flood was very turbulent. The water velocities associated with the horseshoe vortex that formed at the base of the pier fluctuated significantly in direction and magnitude. The 5-degree angle of the flow to pier 3 increased the strength of the turbulence.

The RCI/CSU study also addressed the magnitude of water velocity that would move rocks of given sizes. Their analysis is based on the implicit assumption that the density of all rocks is the same and therefore the weight of all rocks of concern is proportional to the cube of their diameter. Based on their experience, RCI/CSU concluded that the water conditions at the base of pier 3 during flood conditions are most closely represented by studies done in a small turbulent stilling basin to determine the velocities needed to move various size rocks. WJE's hydraulic consultant (Modjeski and Master (M&M)) concluded that conditions somewhere between those encountered in a small turbulent stilling basin and in a non-turbulent stilling basin best represented the conditions that existed at pier 3 at the time of the bridge collapse. Two values were presented in the WJE report, one for a safe weight for placement in the bottom of a channel with turbulent flow and one that was 1.5 times heavier than the other value to account for factors that might allow easier movement, such as exposed placement and rounded shape. Table 7 presents the weight of stone that can be moved by given velocities based on each consultant's experience. Figure 37 is a plot of the velocities as a function of stone size by weight based on RCI/CSU analysis, WJE analysis, and data compiled by the Army Corp of Engineers (Isbash 1932 to 1936).

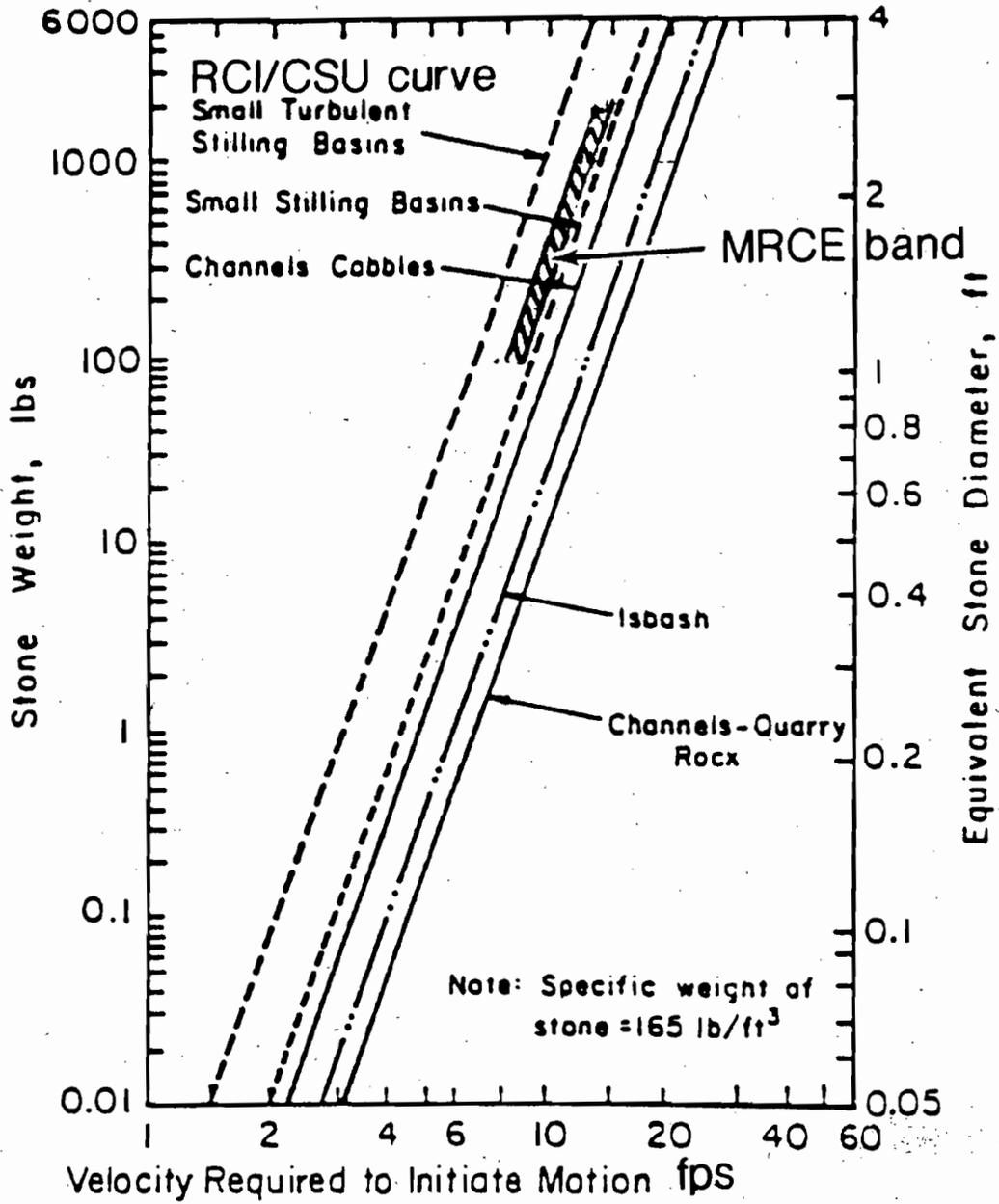


Figure 37.--Critical velocity as a function of stone size.

Table 7.--Critical velocity as a function of stone size (by weight)

Stone weight Lbs	CSU Critical velocity Ft/Sec.	WJE Safe weight Ft/Sec.	Exposed placement (WJE)
100	6.6	9.0	8.4
200	7.4	10.1	9.5
300	7.9	10.8	10.1
400	8.3	11.3	10.6
500	8.6	11.7	11.0
600	8.9	12.1	11.3
800	9.3	12.7	11.9
1,000	9.6	13.1	12.3
2,000	10.9	14.8	13.8
3,000	11.7	--	--
4,000	12.2	--	--
5,000	12.8	--	--

The RCI/CSU study indicated that a velocity of about 8 fps (with a tolerance of  $\pm 2$  fps) could move 300-pound rocks and a velocity of about 10 ft/sec  $\pm 2$  fps could move 1,000-pound rocks.

The RCI/CSU riprap stability analysis concluded that, if the overlaying riprap was removed, the velocity of the turbulent water in the scour hole around pier 3 could have easily scoured out specification 119 bank run material or other types of miscellaneous fill from the excavated area (the area from the top to the bottom of the footings extending 5 feet out from the footings). However, the velocity probably would not have been sufficient to move large rocks (over 500 pounds) from the excavation except where the glacial till had been eroded to form a ramp. According to the RCI/CSU study, the velocities measured in the two-dimensional physical model (7 to 9 fps) would have easily eroded the glacial till and possibly formed ramps parallel to pier 3 on both sides of the pier. The area inside the cofferdam constructed after the accident exhibited some characteristics suggesting that ramps may have been formed at piers 2 and 3.

The RCI/CSU study also concluded that the velocities and turbulence around piers 2 and 3 at the bridge site were large enough to remove any specification 80 riprap placed on top of the excavated area. Because the velocities and turbulence around pier 3 were much larger than around pier 2 (due to the bend in the river), riprap could be removed much faster around pier 3 than pier 2. RCI/CSU concluded that each flow with a peak discharge greater than 30,000 cfs had the potential for removing some of the riprap around pier 3.

MRCE subcontracted with Modjeski and Masters, Consulting Engineers (M&M) to conduct a "simplified analysis of stream hydrology". M&M used a Water Surface Profile (WSPRO) computer program to model flow. Their simplified analysis estimated an

average velocity of 13.1 fps at the bridge with a 63,000 cfs flow in 1987. The velocity at pier 3 reached 15 fps in the M&M model. MRCE estimated that the 300-pound median stone called for in the riprap specification would have been stable except during the four floods that exceeded 40,000 cfs.

Using WSPRO, M&M estimated velocities for various flows as follows:

<u>Flow (cfs)</u>	<u>Velocity (fps)</u>
38,000	9.9
55,000	12.5
63,000	13.1

MRCE estimated that an "average stone size of 1,000 to 1,500 pounds would have been required for 'safe' armoring during a flood of 63,000 cfs in turbulent flow conditions, as occurred during the failure."

#### Cumulative Effect of Floods From 1955 to 1987

RCI/CSU reviewed the flood record for the Schoharie Creek from 1955 to 1987 and found that 38 floods significantly affected the bridge's ability to withstand the 1987 flood. The study concluded that long-term change in the streambed and contraction scour were not factors in the collapse of pier 3. The study also concluded that the cumulative effect of floods between 1955 and 1987 primarily influenced the depth of the local scour and stability of the riprap around pier 3.

#### Other Information

Other Bridges in New York State.-- After the collapse of the Schoharie Creek Bridge, New York State implemented a bridge emergency action plan that required each region in the State to set up teams and perform a "profile check" of bridges in the flood area that had a New York State rating of 1, 2, 3, or 9 under the scour category. The 361 bridges in this category were checked within 72 hours after the program was implemented. Subsequently, New York State conducted a similar profile check of 4,000 bridges statewide, including bridges in the flood area and bridges with ratings of 1, 2, 3, or 9. The profile check included inspections for railing discontinuities, tilt, sag, movement of the substructure, damage to the superstructure from flooding debris, bearing tilt, or indication of damage or scour. As a result, 17 bridges were closed immediately after the flood of 1987, and several other bridges were closed in the following weeks. One bridge was closed due to damage to the deck truss while most were closed due to scour around the substructure.

Similar Bridges in the United States.--At the NTSB public hearing, the FHWA Chief of the Bridge Management Branch in Washington, D.C., testified that he did not know how many bridges in the United States were constructed on spread footings over streams, creeks, and rivers. In addition, he could not specify how many were constructed with two girders or with simple spans. He and the FHWA headquarters were asked on July 8, 1987, and again on August 27, 1987, to provide a nationwide estimate of the number of bridges of a design similar to that of the Schoharie Creek Bridge, that is, those with spread footings and a two-girder non-redundant structure in which each span is simply supported. On September 2, 1987, the FHWA's Chief of the Bridge Division responded that they could not provide this data. On the same day, NTSB requested AASHTO to help obtain the same information from at least nine states. An AASHTO representative later indicated that it would be difficult to obtain this information without requiring a manual search of bridge inventories by each member State.

FHWA Research on Scour.--At the NTSB public hearing, FHWA's Chief of the Hydraulics and Geotechnical Branch described FHWA's scour research effort. In the early 1970s, scour meters were evaluated but were found to be unsuccessful due to silting, freezing, or impact forces from debris. Currently, experiments are being conducted to evaluate the mobile use of radar, sonar, and fathometers to detect scour during flooding and during inspections. In cooperation with the USGS, FHWA is currently evaluating stationary and mobile equipment for measuring scour at existing bridge sites.

Recently, FHWA funded the development and improvement of hydraulic computer modeling to predict scour and stream stability at bridge sites. In addition, FHWA is working on the development of a manual that will contain improved design procedures for scour, and plans to issue a technical advisory on scour sometime in the summer of 1988. Further, the FHWA, in cooperation with the States and the USGS, is developing a nationwide scour study to collect field data on bridge scour using Highway Planning and Research 63/ funds.

Scour Concerns in other States.--On September 7 and 8, 1987, flooding along Goose Creek in southwest Virginia washed out three bridges. In the 1987 flood, flood gages were inundated, but the preliminary estimate was that the flow was about 45,000 cfs, a 100-year plus storm. 64/ (The previous high flow was 25,600 cfs.) NTSB staff conducted a 1-day field trip to view the bridges.

63/ Highway Planning and Research funds are FHWA funds distributed to the States for their purposes. Sometimes States pool these funds for a common project.

64/ Less than one storm of this magnitude is expected to occur in 100 years or more.

The Virginia State Route 747 bridge, built over Goose Creek in 1979, was about 152 feet long and had three stream piers supported by spread footings on solid rock; the abutments were supported on steel H-piles. The roadway approach to the bridge before the abutments was washed out and an area 28 feet wide and 10.3 feet deep was eroded on the east approach. Similar damage occurred on the west approach. The steel piles under the abutments were exposed and most of the layered riprap that had been on the embankment surrounding the abutments under the bridge was washed away.

The Virginia State Route 732 through-truss bridge, built in 1938, was downstream from State Route 747. It was 145 feet long and had a center span 100 feet long. When this truss span failed, it pulled away from the roller bearings, and was pushed to the north bank by the moving water. The south masonry wall, which supported a 26-foot-long span, was also washed down stream.

Further downstream, in Huddleston, Virginia, much of the State Route 626 bridge, built in 1928, was also destroyed. This bridge was about 351 feet long and 17 feet wide, and had eight spans and seven piers. Span 1 remained in place. The footings for piers 2 and 3 were visible and leaning downstream while pier 4 was not visible. The horseshoe vortex scour was observed at the base of several piers that remained standing, including piers 1, 5, and 6. At pier 1, scour was 2.5 feet deep below the streambed, and the footing at the upstream end of pier 2 was completely exposed.

Five other smaller bridges in Virginia collapsed in the storm. At the request of the Safety Board, Virginia searched their bridge inventory file to determine the number of bridges over water with redundant and non-redundant structures and simple spans on substructure units with spread footings. The following figures were obtained. 65/

65/ Similarities to the condition that existed at the Schoharie Creek Bridge are limited by the following limitations of the inventory:

Although a spread footing was indicated, Virginia had no way to determine if that footing would be affected in times of flooding.

Although a simple main system could be identified, there was no way to determine if that span was being supported by the substructure unit that had the spread footing.

Since the code that identified a bridge that had fracture critical details was used to determine if the structure was redundant or non-redundant, there was no way to determine if a non-redundant span was being supported by the substructure unit that had the spread footing when the structure consisted of both type spans.

<u>Type of Bridge</u>	<u>Number</u>
State-maintained highway bridges longer than 19 feet	9,603
State-maintained highway bridges longer than 19 feet and over water	7,574
Redundant bridges longer than 19 feet with spread footings	4,448
Nonredundant bridges longer than 19 feet with spread footings	428

Minnesota provided the Safety Board with data on their bridges with spread footings and possible scour problems. The data indicated the following:

<u>Type of Bridge</u>	<u>Number</u>
Bridges with spread footings	518
Bridges with type of footing unknown	960
Bridges with scour or possible scour	456

#### Other Scour Research

Highway officials of the State of Virginia indicated that they are currently involved in two research projects dealing with scour. As a result of a flood in November 1985, data was gathered on four bridges to verify or modify models used to predict hydraulic characteristics of streams. A report on the findings is being drafted. The other project involves collection of scour data with Maryland, Delaware, and USGS.

In cooperation with the USGS, the FHWA has begun several scour research programs, including pilot studies to evaluate scour prediction equations, projects to develop and evaluate techniques and devices for measuring scour at bridge sites, and research to evaluate the performance of bridges during floods. The USGS is conducting the scour research in States in which cooperative scour data-collection studies are currently underway or are planned. The pilot studies will evaluate at least thirteen scour-prediction equations at selected bridge sites. Several scour detection devices, such as radar, tuned transducers, seismic devices, color photometers, and fathometers with varying potential are being tested. In addition, the FHWA has contracted with the USGS to conduct 120 post-flood inspections of bridges damaged by floods. The USGS will evaluate the performance of bridges during floods, collect on-site measurements of scour, and identify design changes that will improve the stability of bridges in flood conditions. These research projects are not scheduled to be completed until 1992.

The Safety Board's hydraulic consultant at CSU testified at the Safety Board hearing that he was aware of a 3-year study being conducted in Arkansas. Twenty gages placed at locations throughout the State are used to gather routine and flood flow measurements for hydraulic variables as well as scour measurements.

Proposed Rules.--On April 7, 1987, the FHWA issued a notice of proposed rulemaking (NPRM) for revisions to the NBIS. The revised regulation would permit an increase in the maximum 2-year inspection frequency for certain types of bridges, and require identification of bridges that have fracture-critical members 66/ or that warrant underwater inspection or other special inspections. Revisions would also permit bridge inspection team leaders to be certified as competent if they have received Level III certification as bridge safety inspectors under the provisions of the National Institute for Certification in Engineering Technologies. The proposed regulation would require that inventory data on newly load posted, as well as modified or newly completed bridges, be entered into a State's record within 90 days. The FHWA received a total of 61 comments (including comments from the Safety Board) 67/ on the NPRM and is in the process of evaluating the comments and drafting a final rule.

## ANALYSIS

### General

Physical evidence and witness observations indicate that pier 3 and spans 3 and 4 collapsed into the Schoharie Creek during peak flood conditions about 1045 a.m. on April 5, 1987. Videotape taken at the bridge site indicates that about 90 minutes later, pier 2 and span 2 collapsed.

The performance of the passenger car drivers and truck driver during the collapse did not influence the outcome of this accident. Two of the drivers were crossing spans 3 and 4 at the time of the collapse and had no opportunity to take evasive action. Since visibility was reportedly good on top of the bridge, the other three drivers may have observed the falling vehicles and bridge structure. However, these drivers probably would not have had time to react, execute a braking maneuver, and stop their vehicles before reaching the bridge void if their vehicles were traveling at posted highway speeds on wet pavement.

66/ According to the AASHTO definition, fracture-critical members or member components are tension members or tension components of a bridge whose failure would be expected to result in the collapse of the bridge.

67/ See appendix G for NTSB response.

## Failure Modes

An inspection of the spread footings on piers 2 and 3 after the collapse revealed that they had separated into two large pieces and that the soil beneath them was extensively eroded. Survey measurements performed by Wiss, Janney, Elstner Associates, Inc. (WJE) indicated that the south end of pier 3 had tilted and dropped vertically about 5 feet into a large scour hole. This physical evidence suggested that the scour extended far enough beneath both piers to jeopardize the structural integrity of the piers and cause them to rupture. However, it was also possible that the scour holes at piers 2 and 3 developed after the collapse of the bridge as a result of changes in the flow of the creek from the bridge debris that had fallen into the creek. Therefore, the Safety Board examined the possible failure modes that could have produced the pattern of damage observed at the site. These included failure of the bridge superstructure members, including the bearings, from a preexisting deficiency or excessive overload; failure of the substructure from excessive loads; and failure of the bridge substructure from scour of the foundation, which undermined the spread footings.

Mode 1: Failure of the superstructure members, including the bearings.--After the collapse, the girders and bearings were examined to determine if the damage existed before the collapse or was a result of the collapse. The ends of several of the main girders were found bent or highly distorted. This pattern of damage was not typical of preexisting damage, like corrosion or fracture, but instead was more representative of overstress damage, which would have been sustained as the members struck the streambed.

The only damage noted in the fixed and rocker bearings of the bridge was the rounding of the bearing end edges. This condition probably occurred as the spans and girders slid and rotated on the bearings during the collapse sequence. Again, there was no evidence to indicate that this condition existed before the collapse.

No evidence indicated that the bridge sustained excessive vertical or lateral loads on the superstructure before the collapse. There was also no evidence of a corrosive or mechanical failure in any component of the superstructure. Thus, based on a review of the bridge wreckage, the Safety Board concludes that the collapse sequence did not result from a preexisting deficiency in or an overstress of the bearings or other superstructure components.

Mode 2: Excessive loads on the bridge substructure.--Excessive hydraulic forces generated by high velocity flood waters may have exerted excessive loads on the bridge substructure, causing one of its critical members to fail.

If the hydraulic force of the water or debris in the water had exerted excessive loads on the substructure, it is likely that the concrete piers would have fractured into numerous pieces and smaller portions of the cracked piers probably would have been carried downstream by the flood waters. However, the examination revealed that the piers simply ruptured and no parts of the ruptured piers were missing. Further, WJE concluded from its structural analysis that stresses from the weight of the bridge and traffic, combined with the lateral loading from the flood, were not great enough to cause the rupture that occurred in the plinth at pier 3 when it was fully supported by the soil. Calculations from the Resource Consultants, Inc./Colorado State University (RCI/CSU) hydraulic study also indicated that the lateral forces from the stream flow were far too small to affect the massive pier structure. Therefore, the Safety Board concludes that excessive loads on the substructure did not initiate the collapse sequence.

Mode 3: Scour beneath the spread footings undermining the substructure.--After examining the location and position of the bridge components, WJE concluded that pier 3 had lost support before the collapse. WJE further concluded from its examination of the dewatered accident site that the scour condition at the upstream end of pier 3 did not change significantly after the collapse because the fallen main girders of spans 3 and 4 dammed the creek and protected this area. WJE concluded that a deposit of alluvium on the east side of the creek about 200 to 300 feet upstream of pier 3 was evidence that the flow had lost energy in that area of the creek.

WJE stated that a portion of the scour pattern around pier 2 (which had been severely undermined) appeared to be closely related to the location of the bridge components on the streambed. WJE also concluded that some erosion at the upstream end of pier 2 probably occurred before the initial collapse of the bridge, but that the collapse of spans 3 and 4 into the water redirected flow toward the east side of pier 2 and the west bank, which accelerated the scour at pier 2.

The hydraulic flume simulation tests conducted by RCI/CSU corroborated this collapse sequence. The three-dimensional model demonstrated that under conditions simulating the flood of April 1987, local scour occurred initially around pier 3, where it increased until about one-third to one-half the footing was undermined. By the time the footing at pier 3 was substantially undermined in the flume, only limited scouring had taken place at pier 2, and none of the footing had been undermined. However, when simulations of debris from spans 3 and 4 were placed in the flume containing the three-dimensional model, the scour around pier 2 increased rapidly. The increase in scour activity was attributed to a redirection of the flow by the simulated bridge components. The scour at pier 2 then rapidly undermined the footing under pier 2.

Witnesses who observed the collapse of spans 3 and 4 said they heard a loud noise that sounded like an explosion. The videotape of the span 2 collapse recorded a similar noise, which occurred just seconds before each span fell into the creek.

The Safety Board believes that the loud noise was the sound of the concrete plinth at pier 3 rupturing into two separate pieces, amplified through the steel superstructure. After the plinth ruptured, the columns and plinth of the pier no longer acted as an integrated rectangular box structure. The upstream portion of the ruptured plinth rotated and dropped into the scour holes, which caused the collapse of the south column at pier 3 along with the bearings that supported the southeast corner of span 3 and the southwest corner of span 4. Because the bridge spans were noncontinuous and supported by two girders that were simply supported by columns, and because the bearings were tall, the superstructure could not tolerate large movements of the substructure. Thus, because of the design of the bridge, when the upstream portion of the pier 3 plinth dropped into the scour hole, spans 3 and 4 fell suddenly and catastrophically into the creek providing no warning to motorists of the impending collapse.

Based on physical evidence at the bridge site, the physical model, witness statements, and videotape documentation, the Safety Board concludes that scour undermined pier 3 to the point that the pier ruptured. The south portion of the pier dropped into the scour hole, causing the south column at pier 3 to collapse suddenly. As a result, spans 3 and 4 fell into the creek to the south. Bridge debris redirected the flow toward pier 2, accelerating scour at that location and causing the later collapse of pier 2.

### Adequacy of Bridge Design and Construction

The Schoharie Creek Bridge was generally designed and constructed to comply with the 1949 AASHO Standards as required by the design contract. However, there were some ambiguities in the design plans and specifications and some minor deviations in construction from the plans, which, in combination with each other, appear to have contributed to the collapse. The following sections will discuss the aspects of the design and construction of the foundation and substructure that may have contributed to the failure of the bridge, and the aspects of the design of the superstructure that led to the rapid and catastrophic nature of the collapse. Pertinent AASHO Standards (1949) are referenced.

Stability of Substructure.--In designing the Schoharie Creek Bridge, the designers had several options for protecting the substructure against the hydraulic forces of flood waters and the effects of scour. These included using piles under the piers, with construction of the footings at a sufficiently low elevation, protection of the footings with riprap or steel sheeting, or both. This section describes the factors that should have been considered in designing this bridge to ensure stability of the substructure.

Hydraulic study and foundation design.--Section 3.1.1 of AASHO described the process for determining the waterway area (the area under the bridge) and called for a careful study of local conditions including flow [discharge] and frequency, performance of other bridges in the vicinity, and other information pertinent to the design of the bridge and likely to affect the safety of the structure. In response to written questions from the Safety Board, the bridge designer, Pavlo, stated that he did not study the history of Schoharie Creek before preparing the final design. Madigan-Hyland Consulting Engineers (M-H) did conduct a limited hydraulic review as indicated by its hydraulic sheet. However, the sheet did not call for comments nor were comments added concerning the creek's flood history or the performance of structures along the creek during prior floods, even though some of the information was readily available at the time. DPW-DE subsequently provided such information on some floods to M-H.

Correspondence between M-H and DPW relating to hydraulics usually addressed the length of the bridge and the elevation of the backwater, but not the frequency and magnitude of previous floods or their effects on other structures over the Schoharie Creek. For example, Safety Board investigators were unable to find in the M-H hydraulic sheet any mention of the three floods that exceeded 50,000 cfs, which occurred during the first half of the 20th century, let alone an analysis of their importance to the design and construction of the bridge.

The M-H hydraulic sheet does indicate that M-H was aware of the potential for erosion in the banks and the streambed of the Schoharie Creek, but its failure to review the available history limited its appreciation for the potential for scour at this bridge site. If M-H had visited some of the other structures along the creek, such as the aqueduct 3,000 feet north of the bridge, it probably could have observed scour near the piers and this may have heightened its concern for scour.

Such a review would also have revealed that a number of structures along the Schoharie Creek had been built on piles: piles were driven at the aqueduct in 1845, at the railroad bridge in 1905, and at the 5S bridge in 1929. This, in combination with a study of the history of damage done to bridges and other structures by previous floods of the Schoharie Creek, should have alerted M-H to assess fully the need for and the feasibility of designing the bridge with piles for scour protection. By 1953, a number of improvements had been made in machinery, and thus, driving piles should have been easier than it was for the earlier structures.

The initial hydraulic sheet completed by M-H does contain the only reference to piles found in the records on the design of the bridge. However, this document recommended piles only under the abutments. DPW reviewed this document before Pavlo completed the final design, but none of the other design documents referred to the use of piles under the footings of the piers.

Since the maximum scour depth observed at the bridge site was 9 feet below the bottom of the footing of pier 3, scour piles, which AASHO stated should extend not less than 10 feet below the footing, may have provided the stability needed for the substructure to withstand the scour of the 1987 flood. However, since the Safety Board cannot be certain how deep the scour hole at pier 3 may have become had spans 3 and 4 not fallen when they did, it is not possible to conclude that piles driven in accordance with the AASHO recommendations for pile depths would have prevented the bridge collapse. Certainly, if piles were driven deeply enough, the piers would not have lost their support. Therefore, the Safety Board concludes that had the Schoharie Creek Bridge been designed with piles to protect against scour, the collapse may not have occurred, depending on how deeply the piles were driven below the footings.

However, AASHO specifications and design standards in the early 1950s were not absolutely clear as to when piles should be specified by bridge designers. Section 3.5.1 of the AASHO Standards specified that at locations where unusual erosion might occur and where the soil conditions permitted the driving of piles, piles should be used to protect against scour, even if the safe bearing resistance of the natural soil was sufficient to support the structure without piling. Although section 3.5.1 uses the words "unusual erosion," it did not define these words or provide any guidance on how one would determine if "unusual erosion" might occur.

When the Schoharie Creek Bridge was designed and built, the use of riprap was a recognized means of providing protection against scour and riprap was specified in the contract. Without piles, the integrity of the bridge foundation depended completely on the maintenance of riprap for protection against scour.

Depth of Footings.--Section 3.5.2 of AASHO stipulated that the bottom of the footing of a pier in a stream should not be less than 6 feet below the permanent bed of the stream. Further, section 2.1.2 of the AASHO Standards stated that the elevation of the bottom of the footing, as shown on the plans, "shall be considered as approximate only and the engineer may order...changes in... elevation of footings."

Documentation on this bridge is inconsistent concerning the elevation of the streambed at the piers. According to 1951 soil boring tests and a March 1952 field survey, the elevation of the streambed at the bridge site at that time was 276 to 277 feet. However, the design plans (dated September 1952) showed the elevation of the streambed near pier 2 at approximately 273 feet at the northwest corner of the pier and about 275 feet at the southeast corner of the pier. Further, according to a survey for the final quantity estimates, the ground at pier 2 had an elevation of just below 273 feet. The survey also showed the elevation of the bottom of the footing to have been 270 feet (3 feet below the streambed at the northwest corner of pier 2).

To comply with section 2.1.2 of the AASHTO Standards, the design engineer or the engineer in the field should have modified the design plans to place the bottom of the pier 2 footing at an elevation about 267 feet or lower. This elevation would have placed the bottom of the footings 6 feet below the streambed elevation of 273 feet indicated in the final quantity estimates.

Although the lowest streambed elevation shown in any of the design documentation was 273 feet, the elevation of the maximum scour depth near pier 2 was measured at approximately 265 feet, after the accident. Thus, even if the bottom of the footing at pier 2 had been placed 6 feet below the streambed at an elevation of 267 feet or slightly lower, the collapse of pier 2 probably would not have been averted although it may have been delayed. However, the undermining of the foundation beneath pier 2 occurred only after the creek flow was diverted by the debris from spans 3 and 4; consequently, this design deficiency did not contribute to the initial collapse of the bridge.

Unfortunately, in the early 1950s, no method was available to predict scour depth. Today, methods do exist to estimate the maximum depths of scour conservatively, but these methods may not be as precise as desired. The current AASHTO requirement for spread footings is very similar to the 1949 AASHTO requirements. Based on this collapse, as well as on an improved understanding of hydraulics and an improved ability to predict scour, the Safety Board believes that AASHTO should modify its requirements for the depth of the footings (section 4.4.2.1) and require that the depth be based on estimates of the maximum potential depth of scour at the bridge site, rather than on the existing streambed elevation.

Formulas used by CSU predicted a maximum depth of scour at pier 3 of 26 feet. (The maximum depth observed at the accident site was 15 feet.) It is possible that had the bridge not collapsed when it did, with its debris changing the creek flow, the depth of the scour hole at pier 3 might have increased and perhaps even approached the 26-foot depth estimated by CSU. Obviously, a predicted depth of scour of 26 feet would have justified the use of piles at this site although, to have been effective, the piles would have had to be driven well below the 10-foot depth recommended by the 1949 AASHTO standards.

Piles have been used for the bridge built at the site to replace the collapsed bridge. The piles were driven to bedrock (about 45 to 50 feet below the streambed), far exceeding even current AASHTO standards. Section 4.3.1.2 of the current AASHTO Standards still allows piles to be driven to a depth of 10 feet, as did the 1949 version of AASHTO. However, section 4.3.5.4 of the current AASHTO Standards states, "Subsurface investigations shall be made that will determine the probable depth of scour or flotation of material and the condition of lateral support of the piles." Although this could be interpreted by some designers as

overriding section 4.3.1.2, other designers may simply follow the provisions of section 4.3.1.2. Thus, the Safety Board believes that section 4.3.1.2 should be modified to require that the depth of piles exceed the predicted maximum potential depth of scour.

Sheeting.--Notes on the design calculations for item 82 indicated that a cofferdam of steel sheeting (sheet piles) was to have been driven to an elevation of 268 feet around piers 2 and 3. However, the DPW specification for item 82 allowed cofferdams to be constructed from earth or earth-filled bags, as well as from sheet piling. The DPW specifications also stated that cofferdams were to be constructed to permit excavations to depths up to 3 feet below the foundation elevations shown on the plans--267 feet--and that the contractor was to "remove cofferdams." The bridge builder, Perini, had copies of the specifications, quantity estimates, and design plans, but not the design calculations.

In the top view (south elevation) of the bridge (sheet 60 of the design plans), the designer called for sheeting around piers 2 and 3. The Safety Board believes this may have been specified to eliminate the use of the other materials. However, the centerline profile view of the bridge on sheet 60 did not show the sheeting. Further, although the DPW specifications indicate that the item 82 cofferdams were to be removed, they did not state the ultimate disposition of sheeting. The designer stated in a letter to the Safety Board that he had intended for the sheeting to remain in place. The job engineer told Safety Board investigators that he interpreted the design plans to mean that the sheeting could be pulled, which he decided to do.

The designer's quantity estimates indicated that Item 83ST, Temporary Steel Sheeting, was to be used at piers 1 and 4. The item 83ST specification stated that:

...upon completion of the structure, the Contractor may, at his option, remove the corrugated metal or steel sheet piling placed under this item, or leave the same in place after cutting off the tops at the elevation ordered by the Engineer.

The construction logs indicate that the sheeting was pulled from pier 4 and perhaps reused at pier 3. Thus, although the designer may have intended for the sheeting to remain, the two views of the design drawings conflicted with each other and the specifications did not prohibit removal (and, in fact, specified the removal of cofferdams). Interviews with bridge designers and FHWA officials indicate that contractors place considerable credence on the specifications. Therefore, the designer should have specified "Item 82S," permanent steel sheeting, if he wanted to make certain that the sheeting would not be removed.

If the sheeting had been left in place around piers 2 and 3, it would have provided some additional scour protection for the footings but may not have fully protected the footings. The sheeting may have retarded the movement of riprap or the excavation backfill material at the front of the piers. Similarly, sheeting along the sides of the pier would have impeded the water's ability to lift the riprap or backfill material over the top of the sheeting. Eventually, however, the streambed on either side of the pier probably would have been eroded into a ramp sloping up to the north, and the backfill material would then have been pushed up the ramps and out of the excavation.

In addition, the bottom of the sheeting may not have been placed low enough to protect the piers if scour occurred. Field measurements taken at the accident site show that the streambed at pier 3 scoured to an elevation of 261 feet, well below the elevation of the bottom of the steel sheeting had it been left in place (elevation 267 or higher). Once this scour occurred, the value of the sheeting for scour protection would have diminished. Further, the condition of the sheeting (had it been left in place) is also of concern; corrosion may have reduced its ability to withstand the forces of the water and the debris. Also, backfill would have been moved easily after the sheeting south of the pier was undermined. However, the sheeting would have delayed exposure of the footings and movement of the backfill, and the stream flow pattern around pier 3 probably would have been changed. Therefore, the Safety Board concludes that if sheeting had been left in place, it may have altered the collapse sequence of the bridge structure and may have somewhat delayed the collapse; however, the shallow sheeting would not have prevented the collapse of pier 3.

To have provided significant protection for the foundation, the sheeting would have had to have been driven much deeper. Further evaluation of sheeting and the level of protection it may provide to bridges supported on spread footings is necessary to determine how it can be most effectively used to protect against scour. Currently, CSU and the NYSTA are discussing a project to study this possibility.

### Adequacy of Riprap Protection

Riprap Movement.--Based on pictures and inspection logs, the Safety Board concludes that, as indicated on the design plans, Perini did install a thick layer of riprap from the top of the footing (elevation 275 feet), sloping upward to the plinth to an elevation of about 279.5 feet around piers 2 and 3. This layer of riprap, which included large rocks, protected the pier foundations during the flood of record in 1955 and numerous smaller floods. The 1955 flood of record had the potential to move large quantities of riprap from the front of piers 2 and 3. Although the inspection report for the acceptance of the bridge

on May 31, 1956, did not mention riprap movement, photographs taken on October 30, 1956, showed movement of riprap northward along piers 2 and 3. Various photographs taken from 1954 to 1977 during low water disclosed that some of the rocks had moved northward during that period of time.

Perhaps the best documentation of riprap movement is along the west side of pier 2. Figure 38 depicts the movement of riprap and cobbles to the north along the west side of pier 2. Figure 38 was generated from photographs taken in 1953 (figure 20), 1956 (figure 22), and 1977 (figure 24). The photographic analysis (aided by computers) confirms the downstream movement of rock at pier 2 from 1954 to 1977. (See figure 38.) The same three photographs also show the gradual replacement of large riprap by small river cobbles. Only a few very large rocks (1,000- to 3,000-pound size) are visible in the 1977 pictures.

Photographs taken at pier 3 also show movement of riprap. Photographs taken in 1956 indicate that riprap originally placed at the south end of pier 3 moved northward along the east side of the pier. To the east of pier 3, a 3.5-foot increase in riprap over that specified in the design plans is visible in the 1956 photographs. This represented as much as 65 percent of the riprap that had been originally placed at the south end of the piers, assuming that few cobbles were mixed in with the riprap. This indicates that the south end of pier 3 may have lacked significant protection due to lack of riprap since 1956.

Inspections in 1977 and 1979 indicated that some of the riprap and streambed material had moved at pier 3 and even more had moved at pier 2. Due to the lower water velocities at pier 2, lesser amounts of cobble and other larger material may have been transported into the scour area at pier 2 than into the area around pier 3. After 1977, the frequency and magnitude of floods increased, and the movement of riprap away from pier 3 and its replacement with cobbles and streambed material probably increased.

Further, based on the magnitude of their flows, their direction, and their similarities in velocity, the floods of 1955 and 1987 (as demonstrated by photographic evidence and by the results of the physical models and the computer analyses) had similar erosion capability. The Safety Board thus concludes that had the piers been protected by riprap at the time of the April 1987 flood as they were during the 1955 flood, the bridge probably would not have collapsed.

Depth of Initial Riprap.--Based on observations at the site and on the results of the RCI/CSU physical model runs, the Safety Board believes that riprap was not initially placed to the bottom of the pier 3 footing. Observation at the site after the collapse included the following:

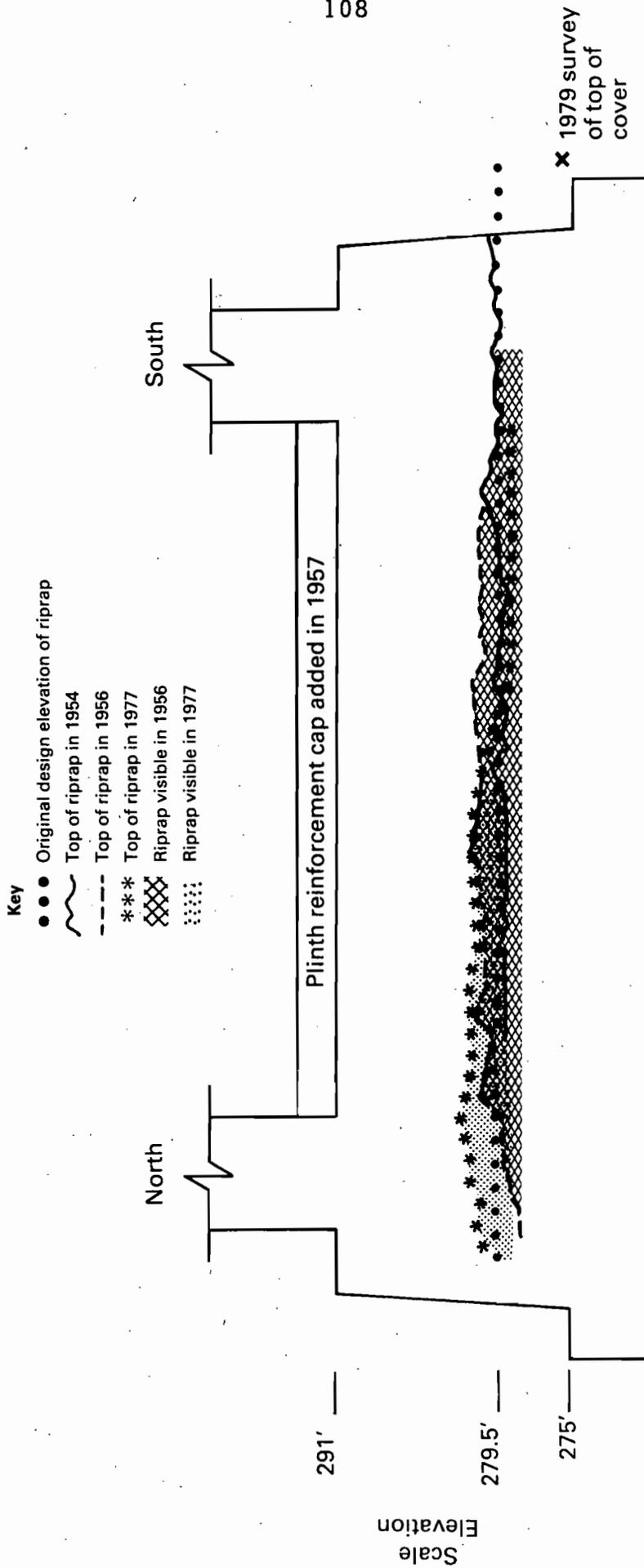


Figure 38.--Movement of riprap along the west side of pier 2.

1. Riprap was not found at the bottom of the footing in those areas at the north end of pier 3 where the footing was covered with rock; in fact, the material found under the rocks was similar to the material expected to be found in the streambed.
2. Drops of paint used on the bridge's superstructure in 1985 and 1986 were found on large rocks at the north end of the pier, indicating that the large rocks probably had not moved during either the 1986 or the 1987 flood.

RCI/CSU made the following observations from the hydraulic physical models:

1. In the three-dimensional physical model, scour at the footings never extended to the north side of pier 3. In some runs, soil actually aggregated at the north side of pier 3.
2. The results of the two-dimensional physical model suggest that the velocities at the lower levels of the scour hole were not sufficient to move the larger riprap from the bottom of the footing, even with the ramp on either side of the pier.

Several reasons were found to explain why riprap was not placed to the bottom of the pier 3 footing during construction of the bridge:

1. Ambiguities in the design plans and inconsistencies in the specifications may have been subject to differing interpretation by engineers in the construction of the bridge.
2. The construction of pier 3 extended late into the fall of 1953 and, with winter approaching, there was a need to backfill the excavated area around the pier as quickly as possible; it was deliberately backfilled with whatever material was readily available.
3. The area may not have been backfilled before the winter of 1953-54 and the floods during that winter could have washed streambed, finger road, haul road, or excavated material back into the excavation around the footing.

Two views in the design plans pertain to riprap placement. One view shows riprap to the bottom of the footing (in the excavated area), and the other shows a thick random layer of riprap over the footings. This ambiguity in the requirements of the plans regarding riprap placement could have been interpreted differently by construction engineers regarding riprap placement.

While the design plans showed random placement of rock in the backfill area, item 80 specified that riprap was to "be placed such that one dimension (the thickness of the riprap) is perpendicular to the prepared bed" and that "the weight of the stone was to be carried by the underlying material and not by the adjacent stones." This type of specification normally requires the careful placement of rocks in layers using cranes, but guided by hand, as was done on the Schoharie Creek embankment slopes. This type of layering of riprap around piers was a common practice on New York State bridges; however, the rock used was often heavier than that specified by item 80. Photographs from 1954 to 1956 indicate that at piers 2 and 3, at least one layer of riprap had been dumped or randomly placed. The photographs show that the riprap was not smoothly and neatly layered.

If the bridge designer had wanted riprap placed to the bottom of the footing, he could have specified in the design plans "Item 78-Stone Filling" and further specified a weight in excess of 300 pounds, but he did not. The designer could also have clearly specified that the excavated area was to be filled with item 80, riprap. Such ambiguities in the plans and inconsistencies in the specifications may have led to the failure to backfill the excavated area around the footings with riprap.

Pier 3 was the last pier built, with the footing concrete poured on September 14, 1953. The concrete had to cure at least 7 days before work could continue around the piers; usually, full loading of the pier would not take place for 28 days. This may have delayed the placement of backfill. The M-H logbook indicated that construction on the columns and the beam began in October 1953 and continued to November 1953; during this time, there was no mention of the placement of riprap at the piers. Due to approaching winter weather, and the DPW's desire to back fill the area between the concrete footing and the sides of the sheeting "as soon as possible after the concrete footing has set up," the area may have been filled with "miscellaneous" material from around the site.

It is also possible that the excavation around the pier footing was not filled in before the winter. In that case, the floods that occurred during the winter and spring of 1954, which ranged from 8,000 to 18,000 cfs, may have filled the area in with riverbed material. December 1953-January 1954 construction photographs show the top riprap visible around pier 2, but none is visible at pier 3, and there is evidence of flooding in the excavations around piers 2 and 3 with erosion of the finger roads.

The job engineer for Perini stated that he did not recall any re-excavation around the piers after the initial excavation. Since the M-H logbook indicates that riprap was being placed at the "pier footings Schoharie Creek" as late as October 5, 1954, it is possible that a layer of riprap was placed over streambed material that had washed into the excavated area during the preceding winter and spring.

The placement of riprap at the piers was crucial to protecting the piers. Because the waters of the Schoharie flowed from south to north, the velocity and level of turbulence was generally greater at the south end of the piers than elsewhere. Thus, it was critical to provide riprap protection around the south end of the piers. Further, because the flow was concentrated on the east side of the channel, stream velocity was greatest at pier 3; therefore, the most critical location for riprap protection was the south end of pier 3.

Because the bridge was designed with spread footings and without piles, riprap protection against scour was essential to the survival of the bridge during floods. Backfill of the excavated area around the footings with material other than riprap made the piers more susceptible to scour than if the excavated area had been backfilled with riprap. Consequently, the Safety Board concludes that the failure to back fill the excavated area with riprap contributed to the collapse of pier 3.

Adequacy of Steel Reinforcement to Prevent Cracks in the Plinth.--The Safety Board carefully analyzed the WJE limited review of the adequacy of the structural design of the bridge, and the WJE detailed study of the ability of the structural design of pier 3 to withstand the flood and the scour. Among the conclusions reached by WJE was that the initial steel reinforcement placed in the top of the plinth (.13 square inches per foot) was the minimum amount required by AASHO (.125 square inches per foot) to resist the formation of temperature and shrinkage cracks.

Toward the end of the first year of the bridge's service, vertical cracks were observed in the mid-region of the plinth of all four piers. According to WJE, the cracking was primarily due to tensile bending stresses in the plinth from the relatively uniform distribution of soil pressure below the footing. Loads due to temperature changes, wind, ice, vehicles, etc. may also have increased the stresses in the structure.

In the design calculations, the weight distribution of the pier over the entire footing area was calculated to determine the size of the footing area that would be necessary to support the bridge and provide a loading on the soil of no greater than 2.5 tsf. The designer also calculated the amount of steel reinforcement required at the bottom of the footing to resist stresses caused by the column loads that were distributed to the footing. This calculation complied with the 1949 AASHO guidelines. However, the original calculations apparently did not consider the effect on the plinth of soil pressure beneath the footing. AASHO stated that "in plain concrete footings, the tensile stress shall be computed on the basis of a monolithic section...." The Safety Board believes that it would have been appropriate for Pavlo to consider the footing and plinth as

a monolithic section in resisting the soil pressure. 68/ Thus, he should have calculated the amount of steel reinforcement required in the top of the plinth to resist the tensile stresses that could have been created by the upward forces of the soil on the footing and plinth as a monolithic section.

Based on a two-dimensional skeletal frame model of the bridge, WJE computed tensile stress in the top of the plinth (prior to the installation of the plinth reinforcement cap in 1957) to have been about 140 psi in piers 2 through 4, and 110 psi in pier 1. (The stress was less in pier 1 because the plinth section was 3 feet deeper than in the other piers.) Because the maximum allowable stress in tension based on the AASHTO design conditions (plain concrete with a compressive strength of 3,000 psi) was 90 psi, the tensile stress at the top of the plinths exceeded this value, and therefore the steel reinforcement of 0.13 square inches per foot was inadequate. In a November 21, 1955, letter to NYSTA, DPW-HQ indicated that, in the preliminary layout, M-H initially specified 40 to 50 square inches per foot of reinforcing steel in the top of the plinth, but this amount of reinforcement was not supplied. The plinth would not have cracked in 1955 if it had been reinforced according to the preliminary layout. (It may also not have cracked in 1987, but the footing may have tilted into the scour hole in one piece, still causing the bridge to collapse.) Further, finite element computations made by WJE showed that under gravity loading, the maximum tensile flexural stresses (125 psi) on the pier as originally built also exceeded the design limits. This probably caused the plinth (without the reinforcement cap) to crack in 1955.

The plinth reinforcement caps, which had 53.3 square inches per foot of reinforcing steel, were designed to repair the vertical cracks in the plinths of all four piers. However, the plinth reinforcement caps were not anchored to the adjacent columns by doweling or straps. M-H initially considered attaching the caps in this manner, but decided against it due to the possible damage that could have occurred to the existing steel reinforcement in the columns by the doweling process.

Because the plinth reinforcement was not anchored to the columns, the plinths were more vulnerable to failure at the juncture of the cap and the column when the footings were undermined from scour. Paragraph 3.7.7(e) of the 1949 AASHTO specification stipulates that structural reinforcement shall be

68/ The plinth was fastened to the footing with dowels and it therefore acted as a monolithic structure. Since there was only very light steel (for temperature and shrinkage) in the plinth, it can be considered "plain concrete."

extended to other supports to provide for unanticipated distributions of loads, shifting points of inflection, yielding of supports, or other possibilities. Although the repair restored the integrity of the plinth at the location of the 1955 vertical crack, it was not a fully satisfactory repair because the failure to attach the cap to the columns omitted an available level of structural support.

From the finite element structural analysis, WJE concluded that when scouring extended 25 to 30 feet in length beneath pier 3, the tensile stress at the upper surface of the unreinforced plinth (between the inner face of the north column and the first dowel connecting the reinforcement cap to the plinth) exceeded the estimated tensile strength of the "lightly" reinforced concrete. Consequently, the plinth fractured in that location and the crack propagated from the upper surface of the plinth, at approximately a 25-degree angle from the vertical. When the crack reached the preexisting vertical crack closer to the center of the plinth, the plinth separated into two pieces. This corresponds closely to the crack propagation observed in the plinth after the collapse of the bridge. WJE further concluded that to have adequately reinforced the area of the plinth where the fracture initiated, the plinth reinforcement cap would had to be connected to the column.

Although the inadequacy of the repair of the crack was a key factor in the rupture of pier 3, it is uncertain whether a better repair would have prevented the collapse of pier 3 at a later time. If the plinth reinforcement elements had been connected to the columns, or if the pier had been adequately reinforced initially, the undermining of the footing might have continued until pier 3, acting as a rigid body, became unstable and rotated slowly into the scour hole. It is possible that under these conditions, a misalignment of the upper deck may have provided some early warning of the impending collapse, but, given the nonredundant nature of the structure, this is doubtful. Thus, the lack of sufficient steel reinforcement in the design of the plinth and the improper repair of the plinth probably did not contribute to the collapse of pier 3, and may not have contributed even to its suddenness.

Critical Design Features of the Bridge.--A combination of superstructure design features contributed to the nonredundancy of the Schoharie Creek Bridge. The simply supported two-girder superstructure was designed with noncontinuous spans, which prevented transfer of loads from one span to another in the event of a structural failure affecting a span. Had the superstructure been built with continuous spans, it may have delayed the collapse of spans 3 and 4 momentarily and may have provided some advance warning to motorists on the bridge. However, a continuous span superstructure probably would not have been able to transfer, indefinitely, the entire load previously supported by pier 3. Thus, spans 3 and 4 would have eventually collapsed.

In addition, the superstructure was mounted on relatively tall bearings, which could not tolerate even relatively small amounts of movement of the superstructure before becoming unstable and possibly overstressed. If such a bearing becomes unstable due to excessive movement, the bearing may overturn or twist. Also, if a bearing is overstressed, the bearing can fracture or bend.

A combination of substructure design features also contributed to the bridge's susceptibility to failure. As already discussed, the originally designed and constructed plinth lacked adequate steel reinforcement to compensate for tensile loading in the plinth, and the reinforcement cap was not permanently anchored to the pier columns to make a more rigid substructure. In addition, the bridge was designed without piles. Although piles were not needed to support the soil bearing loads of the bridge, they would have provided support after the soil beneath the spread footings was eroded by scour.

Because of the design of these critical features, the bridge was more susceptible to the failure it suffered during the April 1987 flood. The lack of structural redundancy in the design of the bridge contributed to the very rapid and catastrophic collapse following the undermining of pier 3.

The Safety Board previously addressed the issue of redundancy in its final report on the Mianus Bridge collapse 69/, where it stated:

While the design of the suspended span of the Mianus River bridge that collapsed was not redundant, redundancy was not a specified design consideration when the bridge was designed in 1955. Indeed, redundancy is not required even today. However, the 1977 AASHTO "Standard Specifications for Bridges" require a reduction in the allowable range of stress in structures subject to repetitive loadings where there is nonredundant load path structures; i.e., "structure types with a single load path where a single fracture can lead to catastrophic collapse." The Safety Board is concerned that the concept of redundancy is not well-defined and that disagreements among experts as to what is meant or intended by redundancy have not been resolved. The concept needs to be clarified in the interest of the safety of future designs.

69/ For more information, see Highway Accident Report--"Collapse of a Suspended Span of Interstate Route 95 Highway Bridge Over the Mianus River, Greenwich, Connecticut, June 28, 1983 (NTSB/HAR-84/03).

The failures of the Silver, Mianus, Chickasawbogue, and Schoharie bridges emphasize the importance of identifying and periodically inspecting a bridge's critical features--that is, those features whose weakening or failure can create conditions for a catastrophic collapse. These critical features should receive greater attention than other features. Also, critical features should be highlighted on design and as-built plans to make certain that State and local highway officials take greater care in inspecting and maintaining them. This need is especially important for structurally nonredundant bridges, where the failure of a critical feature could cause not only the collapse of a bridge, but could also result in a rapid, catastrophic collapse like that which occurred to the Schoharie Creek Bridge.

On April 7, 1987, the FHWA published a Notice of Proposed Rulemaking (NPRM) to amend the NBIS to require, among other things, that the person responsible for bridge inspections prepare and maintain a master list of the bridges with fracture-critical members (identifying their fracture-critical members), the bridges with underwater members that cannot be examined visually or by touch (describing their underwater members), and the bridges with unique or special features requiring additional attention during inspection. The master list is also proposed to include the date of the last inspection of these critical features and a description of the findings and any followup action required. In its comments on the NPRM, the Safety Board stated that it was concerned about certain other proposed rule changes and asked that the FHWA await making certain changes until after the investigation of the collapse of the Schoharie Creek Bridge had been completed and the causes of the collapse were thoroughly understood.

The proposed requirements to expand the information contained in the master list of bridges would be a significant step forward. However, the Safety Board believes that the FHWA should include on the master list the type of scour protection provided (piles, sheeting, or riprap) to the bridge, along with such critical features as noncontinuous spans, twin girder design, and bearings that cannot tolerate any significant misalignment.

As previously discussed, the general characteristics of the Schoharie Creek Bridge were similar to many bridges constructed in the late 1940s to 1960s. Many of these bridges will be rehabilitated within the next decade. During these rehabilitations, foundation and superstructure designs should be carefully analyzed to determine whether redundancy or additional protection is warranted and can be built into these structures. For example, it may be feasible to drive steel sheeting around the piers of bridges not founded on piles.

The Safety Board also believes that when new bridges are built over water or when the foundations of older bridges over water receive major structural renovations, designers should provide for scour protection, if significant flooding is likely and if the streambed is susceptible to erosion. Options in providing such protection include placing footings deeply enough below maximum potential scour depths, placing scour piles, driving sheeting deeply, or other similar improvements.

The Safety Board also believes that designers should perform comprehensive hydrologic and hydraulic studies to determine the potential for scour, including the use of analytical methodologies for predicting the potential maximum depth of scour. The use of spread footings with riprap as protection against scour where significant erosion is a possibility should not be considered a safe design practice. FHWA guidance and AASHTO specifications should be modified to reflect the lessons learned from the catastrophic collapse of the Schoharie Creek Bridge.

#### Adequacy of Bridge Inspection and Maintenance

General.--A review of the NYSTA maintenance records for the Schoharie Creek Bridge indicated that the bridge received regular maintenance such as painting of the superstructure, patching of the deck, and the sealing of joints. After the collapse, observations by Safety Board investigators of the remains of the deck, steel beams and girders, bearings, columns, and piers corroborated this. The maintenance records, however, did not include any entries concerning the maintenance of riprap around the piers.

Bridge Inspection and Maintenance.--From the time the Thruway was opened, the NYSTA, as did NYSDOT and many other organizations, used maintenance personnel to inspect bridges for both maintenance needs and safety inspections. The inspections in the Albany division were accomplished by not by engineers, but by personnel whose primary responsibilities were in bridge maintenance. The Albany assistant division engineer (bridges) was not a professional engineer but had received the training and had the years of experience required by the NBIS to qualify for conducting bridge inspections.

However, in his 1986 inspection of the bridge, and in previous inspections, he failed to evaluate the condition of the riprap at the piers properly, and he failed to take the dropline readings necessary to evaluate the conditions in the streambed. These two tasks were specifically required in the BIM-82 and earlier documents on bridge inspections. The fact that he overlooked these two tasks indicated that he either did not think they were important or he did not understand their importance. In addition, the assistant engineer's supervisors, who should have reviewed his reports, apparently did not review his reports or failed to recognize the seriousness of the omissions and therefore did not attempt to correct the situation.

The Albany assistant division engineer (bridges) may have assumed that the bridge was built on piles and therefore did not regard riprap maintenance as important. In his 1986 inspection of the bridge, he gave piles a rating of "9," indicating condition unknown, rather than "8," not applicable. At the Safety Board's public hearing, he also indicated that he thought the bridge was constructed on piles. Some of the bridge inspection reports that he signed as far back as 1970 indicate that he thought the bridge was built on piles, but other reports indicate the opposite.

Entries in the maintenance log of the Schoharie Creek Bridge date back to 1955. None of the entries address the maintenance of riprap. The Albany assistant division engineer (bridges) said that he did not recall riprap ever having been placed or maintained around the pier footings. Further, there is no evidence to indicate that riprap had ever been replaced around the piers after the bridge was opened to traffic in 1954.

In 1979, Seelye, Stevenson, Value, and Knecht (Seelye) conducted bridge inspections for NYSDOT to comply with the NBIS inventory requirements for off-system bridges. Sketches made by the assistant team leader during that inspection clearly showed that riprap around piers 2 and 3 was missing. According to the sketches, there was apparently no riprap around the plinth at the upstream end of pier 2, even though the original plans called for 4.5 feet of riprap above the top of the footing adjacent to the plinth. The team leader wrote on the sketch, "5' - 4" of cover" above bottom of the footing at the upstream end of the west face of pier 2. Since the footing was 5 feet thick, there could not have been more than 4 inches of material covering the top of the footing. Thus, it is apparent that there was no riprap at the upstream end of the footing. The 4 inches of material shown on the sketches was too thin to have been riprap because the riprap was specified to be a minimum of 8 inches. The 4 inches shown was probably small rock or soil. The assistant team leader, who drew the sketches, stated that he could not recall if riprap was present. The assistant had drawn in "scattered stone" on the downstream end of the west side of pier 2 but did not recall its exact extent or shape; he said that it was large stone of at least basketball size. It may have been cobbles or riprap.

The streambed adjacent to pier 2 and at the downstream end of its east side was drawn lower than the streambed upstream. The entire east side above the top of footing was shown under water; no rock was shown. From photographs taken by the assistant team leader, it was estimated that the water height was about 3.5 feet above the top of the footing. Riprap was obviously missing.

The measurements on the sketches also indicated that riprap was missing at pier 3. The assistant's sketch of the west side showed that there were 20 inches (rather than the 4.5 feet specified in the plans) of cover at a point 25 feet downstream from the upstream nose of the plinth. Apparently, riprap was

missing at least near the top of the footing. The elevation of the bottom of the footing was noted on the sketch as 270.0, the design elevation of the bottom. The assistant said that this had been inked in by the team leader. To note this correct elevation, the team leader must have reviewed the design or as-built plans. If he had reviewed the plans, he should have noted that riprap was required above the height measured by his assistant.

The east face of pier 3 showed material 2 feet 4 inches above the top of the footing at a point 4 feet 6 inches from the upstream nose of the plinth. Toward the downstream end of the footing, there were about 6 feet of material over the top of the footing or about 1 1/2 feet above the level of riprap required by the original design plans. The Safety Board's interpretation of the photographs indicates that the stone was probably a mixture of riprap and large river cobble, with more of the latter.

These measurements, when compared with the original design plans, showed a significant decrement in the riprap cover of the footing. The measurements and photographs clearly indicated that riprap was not piled at an even level around the plinth. This information should have alerted a person knowledgeable in river mechanics and structures that riprap had moved, posing a danger to the structure. However, the team leader, a registered professional engineer, gave both piers 2 and 3 a rating of 6 for its scour condition. This was the best rating that could be given if erosion or scour had affected, in any way, the material above the bottom of the footing but had not undermined the footing. A rating of "7" would have indicated that there had not been any loss of material around the piers.

The team leader also coded the pier-piles column in the bridge inspection report as "8," meaning that no piles were present under the piers.

The Safety Board believes that the sketches showed that a significant amount of riprap had moved away from the upstream ends of the piers in 1979 and, especially since there were no piles, Seelye should have, in accordance with its agreement with the NYSDOT, immediately called the NYSDOT project manager to alert him. The call also should have been followed with a letter. However, there is no evidence that the firm so notified the NYSDOT (or the NYSTA) of the riprap deficiency.

When the NYSDOT received the report, it did not notify NYSTA of the missing riprap, indicating either that NYSDOT personnel did not review the report or that they believed the missing riprap required no attention. It is quite likely that NYSDOT personnel did not review the report since they only reviewed some of the inspection reports and those they did review were generally reviewed for coding and format errors only.

When NYSTA finally received the report in April 1980, it did not replace the missing riprap, indicating either that it also did not review the report or that it did not consider the situation serious enough to require correction. If the report was reviewed by the NYSTA, the sketches and the rating elements should have alerted the reviewer that the bridge was not built on piles and that the depletion of riprap was important. Further, the Seelye inspections should have relieved the NYSTA of the need to perform a bridge inspection that year; the time saved could have, and should have, been used to thoroughly analyze the Seelye report. (Seelye's inspections of the Schoharie Creek Bridge were on March 26 and August 15, 1979; the NYSTA's inspection was on October 21, 1979.)

The major rehabilitation project completed in 1982 greatly improved the superstructure and substructure above the water line. Unfortunately, the plans finalized by NYSTA did not call for the replacement of missing riprap with 600 cubic yards of 600-pound riprap, as had been specified in reports and plans prepared by Dale Engineering, Inc. (Dale). Replacement riprap was removed from the plans at the direction of the NYSTA technician responsible for finalizing the plans.

Memoranda written in 1978 and 1980 by NYSTA personnel indicated that the assistant superintendent of maintenance (bridges), the director of construction and design, and the design unit head were aware that riprap had been called for as part of the rehabilitation plans. When the technician decided to delete riprap from the final plans, these same supervisors either checked the plans and agreed with his decision, or they did not check the plans. The Safety Board believes that a failure of the supervisor to review this decision would have been a major deficiency in his oversight of a subordinate. In either case, however, the decision not to replace the riprap as specified by Dale was a critical decision that contributed to the cause of the accident.

The review and analysis of the reports of the NYSTA bridge safety inspections of the underwater portions of the bridge were inadequate. Further, the NYSTA inspectors were not well supervised; their supervisors did not correct them when they failed to note and address missing riprap or when they failed to fill out the underwater section of the forms properly. Further, there was little quality control, especially of the information on the forms relating to the inspection of the underwater elements of the bridges.

These failures may have, in part, resulted from inadequate NYSTA (and NYSDOT) policies and guidelines about when conditions at the foundation of underwater members of bridges warranted maintenance. For example, the NYSTA assistant superintendent of maintenance (bridges), the bridge inspector's supervisor, said that a NYSDOT manual stated that riprap should be replaced before "...scour progresses to a depth dangerous to the stability of a

structure (1/2 of the thickness of the pier footing...)." The Safety Board is not aware of any specific guidance provided by NYSTA to its inspectors about when riprap replacement was warranted. The Safety Board believes that the NYSDOT guidance was not proper and should be substantially modified. (In a December 7, 1987, memorandum, NYSTA directed its employees to delete the reference to 1/2 the footing depth from the NYSDOT manual.)

The Safety Board believes that the inadequate guidance in the replacement of riprap provided to the NYSTA inspectors resulted, in part, from the lack of specific guidance available at the time from FHWA or AASHTO. In fact, it is not clear that the situation is any better today. The Safety Board has reviewed literature from several organizations that provided guidance on bridge inspection and maintenance and has found no specific guidance on when to replace riprap and very little on when to repair scour damage at piers founded on spread footings. Many bridge engineers state that specific guidance cannot be provided, but that inspectors need to use their engineering judgment.

The circumstances of this accident show that better guidance is needed. Inspectors (and some supervisors) from the NYSTA, the NYSDOT, and Seelye either failed to understand the importance of riprap or failed to recognize that sufficient riprap had migrated from piers 2 and 3 to pose a danger to the bridge.

The Safety Board is aware that specific guidance cannot cover every possible condition and that bridge inspectors indeed need good engineering judgment. The Safety Board also recognizes that experienced bridge engineers may generally be able to recognize when riprap needs to be replenished or replaced or when other foundation repairs are required. However, most bridge inspectors are not now, and are not likely to be, experienced bridge engineers. The Safety Board is thus convinced that specific guidance must be provided to bridge inspectors.

The Safety Board believes that additional research is needed to determine the size and amount of riprap needed for scour protection and the degree of depletion that may occur before replacement is necessary. (The Safety Board recognizes that highway maintenance departments cannot replace each rock as it moves.) The Safety Board is concerned that bridges similar to the Schoharie Creek Bridge may not be receiving proper riprap maintenance because there is no proper guidance as to when to replace riprap. Therefore, the Safety Board believes that, until research is done to establish better guidance, AASHTO should provide guidance, and the NYSDOT should modify their guidance to specify that, after each inspection of a bridge that depends upon riprap for scour protection, any missing riprap must be replenished to design specifications or to a higher level of protection.

Riprap must be maintained to prevent erosion of the soil around and beneath the footings. It is highly probable that had the NYSTA maintained riprap of a similar weight and to a similar level as that placed originally, the bridge would not have collapsed. In addition, if NYSTA had followed Dale's plans for replacing the missing riprap (Item 80) with 600-pound riprap, which was twice the weight specified in the original design of the bridge, the riprap would have been more difficult to move and, therefore, would have protected the footings more effectively.

#### Overview by NYSTA's Insurance Carrier

The recommendations made by the Insurance Carrier's construction specialist as a result of his review of the NYSTA's bridge inspection program were quite specific on the need for improvements in the NYSTA bridge inspection program, including inspection and documentation of scour, performance of underwater inspections, provision of equipment for measuring scour, and performance of quality control of bridge inspection reports. NYSTA responses to the recommendations were generally positive, but its implementation of corrective action was not immediate. For example, the insurer recommended underwater inspections within 6 months (or by May 1986) for bridges with substructures hidden from view by water. However, the NYSTA had not even started such inspections 1 year later.

Although pier 2 of the Schoharie Creek Bridge was less accessible than pier 3 for inspection because the water was deeper there, both piers were sufficiently accessible to permit dropline readings, soundings, or probings. While an underwater inspection performed by a diver would have been superior, probing or dropline readings would have been sufficient to determine if riprap was missing. The specialist for the insurance company pointed this out in his recommendation to NYSTA, advising that their inspectors use the methods presented in NYSDOT's Bridge Inspection Manual -82. The NYSTA had been using this document since the early 1980s, even though it did not officially adopt it for use in biennial inspections until October 1986.

The insurer was also concerned with the quality control of the bridge inspection report. In its response, NYSTA agreed that improvement was needed. Apparently, however, NYSTA either did not act to improve its quality control or was unsuccessful since the coding of inspection elements in the inspection reports continued to be inconsistent and incorrect and there was no scour documentation. Both NYSTA and NYSDOT approved the inspection reports, despite their failure to meet the quality control standards specified in NYSDOT's Bridge Inspection Manual - 82.

Although the NYSTA was planning improvements in its bridge inspection program in response to the insurance company's recommendations, it had not implemented them at the time of the bridge collapse.

Since the collapse of the Schoharie Creek Bridge, according to NYSTA, it has accelerated its efforts to inspect its bridges in accordance with the procedures and guidelines of the NYSDOT and the NBIS. The NYSTA has contracted with consultants for the inspection of almost 40 percent of its bridges; the NYSDOT is inspecting most of the remainder of NYSTA's bridges. NYSTA has set up a schedule to have the underwater elements of its bridges inspected every 5 years or less. Further, according to the Chief Engineer of the NYSTA, the bridge inspection forms completed by the consultant inspectors are now being reviewed by another inspector. The forms will then be checked again for coding accuracy and for compliance with the inventory requirements of the NBIS by NYSTA, this time by inspectors within a newly formed bridge safety unit within the NYSTA. The forms are also reviewed to determine if the bridge is in need of maintenance.

Despite these and other improvements, the NYSTA has not issued more specific guidance to its inspection and maintenance staff on how to determine when the replenishment or replacement of riprap or other repairs to foundations are needed. It is, however, scrutinizing documents concerning the maintenance of highway structures with a view toward improving guidelines. The Safety Board believes this must be done by the NYSTA.

### Underwater Inspection

Investigation of the collapse of the Chickasawbogue Bridge revealed that many States were not performing underwater inspections of their bridges. Further, the inspections that were being performed were not sufficiently thorough. As a result of the Chickasawbogue accident, the Safety Board issued Safety Recommendation H-86-3, dated June 17, 1986, to the FHWA:

Establish criteria for inspecting the underwater elements of bridges which consider the following factors as they relate to bridge design and maintenance:

Complexity of structure and materials used,  
Marine environment surrounding the underwater  
elements of the bridges, and  
Frequency and magnitude of loads on the bridges.

In response to this recommendation, the FHWA informed the Safety Board that it was preparing an NPRM to revise the NBIS for underwater inspections to address the safety issues listed. (The NPRM was subsequently published in April 1987.) Based on this action, recommendation H-86-3 has been classified "Open-Acceptable Action", pending revision to the NBIS to meet the intent of this recommendation.

The April 1987 NPRM proposes that States identify and maintain a master list of bridges with underwater members that cannot be visually evaluated during low flow or by touch. The NPRM also proposes that underwater inspections must be conducted at least every 5 years. The Safety Board commented on the NPRM to FHWA, stating that the proposed regulations do not adequately respond to its prior recommendation that FHWA establish specific criteria for comprehensive underwater inspections of bridges.

Because of FHWA's failure to oversee and enforce the requirements of the existing NBIS adequately, particularly with regard to underwater inspections, the Safety Board has also indicated that it does not believe that the current requirements for biennial inspections of bridges should be changed as proposed in the NPRM. The Safety Board believes that the streambed around piers of scour-critical bridges should be inspected for erosion during biennial inspection and after floods.

The frequency and extent of underwater inspections should be based on such factors as the characteristics of the stream channel, the velocity of the stream, the propensity of the stream to flooding, the type of footing, and the type of structure. If a stream has a history of turbulent flow, as does the Schoharie Creek, and if the bridge is built on piers with shallow spread footings protected by riprap, more frequent and thorough underwater inspections are required than for a bridge with piers in placid water or on footings set very deeply beneath the streambed and on piles.

Specific criteria for underwater inspections should be established within the NBIS, based on these factors, and not left to the discretion of State highway officials. This will help to ensure that the inspections are comprehensive and frequent enough to account for the complexity of the bridge substructure and foundation, and its environment. In addition, such criteria will promote uniformity among the States in conducting their underwater inspections. Improved guidance for inspectors is needed on what constitutes sufficient deterioration of scour protection (such as riprap) or of the foundation to require that repairs be made. Finally, the NBIS should be expanded to improve guidance on how to inspect other underwater elements, such as scour piles and sheeting.

The Safety Board believes that the circumstances that led to the collapse of the Schoharie Creek Bridge were not isolated events but may represent conditions that can occur at other bridge sites throughout the country. Because the general design of the collapsed bridge was similar to the design of many bridges constructed in the late 1940s through 1960s, there is a potential for other similarly designed and constructed bridges to collapse catastrophically, from erosion of their foundations.

In an attempt to try to quantify the magnitude of this problem, the Safety Board requested the FHWA to determine the number of bridges nationwide over water that are similar to the collapsed Schoharie Creek Bridge. The FHWA responded that it could not provide this data. However, the Safety Board was able to obtain such data from three States (Minnesota, New York, and Virginia) which, as a result of the Schoharie Creek Bridge collapse or other recent floods within the States, had performed data searches. Consequently, several hundred bridges over water with shallow spread footings and/or nonredundant structural features were identified and inspected, and at least 25 bridges (15 after the inspection) were closed, pending repair.

The Safety Board believes that the search initiated within the three States should be extended to States that have not made such a search, particularly in light of recent FHWA data. A 1987 FHWA survey indicated that at least 43,000 bridges nationwide have not been inspected within the last 2 years. More importantly, it is still not clear whether the bridges that have been inspected have received comprehensive underwater inspections. In view of this, the Safety Board believes that the FHWA should require the States to review their bridge inventory data, identify bridges similar to the Schoharie Creek Bridge, and conduct underwater inspections of these bridges as needed.

#### NYSDOT's Overview of NYSTA

In 1978, NYSDOT recognized that NYSTA and other public entities did not meet all the requirements of the NBIS, which was extended by the Federal Surface Transportation Assistance Act of 1978 to include all bridges carrying traffic on public roads (off-system bridges). To meet the Federal requirements, NYSDOT had to inventory 12,000 to 13,000 off-system bridges. As is often the case with new laws, it is often difficult at first to execute them properly because of the tremendous increase in start-up resources required. Because of the immense workload required by the new regulations, NYSDOT hired consultants to inspect many of the Thruway bridges. These inspections (which included the Schoharie Creek Bridge) were a one-time effort by NYSDOT to comply initially with the inventory requirements of the 1978 Act.

While the NBIS is specific in its requirements that the States have a bridge inspection program and maintain an inventory of their bridges, the NBIS does not specify how the States are to implement some aspects of the NBIS. For example, the NBIS does not specify who is to perform inspections of, and supply inventory data for, bridges that carry traffic on public roads that are owned and operated by public authorities or other non-State entities. As a result, in implementing the NBIS, NYSDOT inspected some of the Thruway bridges crossing State and county roads.

Despite the NYSDOT effort to comply with the NBIS by hiring consulting engineers to do bridge inspections, the information that they obtained was used primarily to satisfy items specified in FHWA's Structure Inventory and Appraisal Sheet (SI&A sheet). They did not analyze or otherwise use the results of the inspection of the bridge. Thus, the Safety Board concludes that NYSDOT lost an opportunity to learn about the missing riprap at the Schoharie Creek Bridge and to alert the NYSTA to correct the situation.

At the time of the collapse, the NYSDOT was developing criteria and methods for performing underwater inspections. It had established a list of bridges that were to receive an underwater inspection, which included a list of bridges from the NYSTA; the Schoharie Creek Bridge was on the list. However, the issuance of a contract for the underwater inspections was delayed because, according to the NYSDOT, New York State's share of Federal highway funds was exhausted. Apparently, no NYSDOT official ever notified NYSTA officials of the delay and the NYSTA took no other action. In the meantime, the Schoharie Creek Bridge collapsed.

The Safety Board believes that a proper underwater inspection of the Schoharie Creek Bridge piers before their collapse may have uncovered a lack of adequate riprap or other manifestations of scour, such as a scour hole in the streambed. Such additional evidence of scour may have sufficiently motivated the NYSTA to replace the missing riprap.

Evidence available to the Safety Board suggests that the NYSTA attempted to cooperate with the NYSDOT in its effort to comply with the inventory requirements of the NBIS by providing the NYSDOT with the inspection data it was collecting. However, NYSDOT had no authority to compel NYSTA or any other authority or municipality to cooperate in complying with the NBIS.

As a result of the collapse of the Schoharie Creek Bridge, the New York State legislature is deliberating proposed legislation that generally will require public authorities and other entities owning and operating off-system bridges in the State to inspect their bridges in accordance with the NBIS and provide the data to NYSDOT. However, the passage of this legislation will only provide a limited solution to a larger problem. Numerous public authorities nationwide own bridges that are subject to the NBIS. The laws governing the relationship between the State highway departments and public authorities vary from State to State, but many States may face the same problem as New York, i.e., they do not have the authority to force public authorities into compliance with the NBIS. These States may also need to amend their highway laws to clarify how they will comply with the NBIS.

As a result of the collapse of the Schoharie Creek Bridge, the NYSDOT has also improved its bridge inspection program, formalizing many of its previously unwritten inspection procedures, especially its procedures for inspecting bridges over water. Inspections are scheduled for periods of low water to permit visual inspection of the substructural members and probing beneath the water surface for evidence of scour at the bridge piers, in the streambed, or at the bank. This will also facilitate improved documentation of the foundation and the streambed. When substructural members cannot be visually inspected, then followup diver inspections are required. Bridges with spread footings in water will receive inspections that consider scour susceptibility.

Hydraulic evaluations will be made of bridges over water. This will identify changes in the streamflow velocities, changes in the streambed conditions, both upstream/downstream of the bridge and at the bridge site, and permit the NYSDOT to update the scour susceptibility of the bridge as compared to original design conditions.

Immediately after any flood, the NYSDOT will implement emergency inspections to check bridges in the flood areas for tilt, sag, movement, or other evidence of damage to substructural/superstructure members, and for evidence of scour at the bridge site. If any critical discrepancies are detected, then the bridge(s) will be closed immediately or monitored continuously to see if conditions that would warrant a bridge closure have changed.

The NYSDOT is also evaluating scour detection equipment. Three State inspection teams of two persons each have been added to bring the total to 26 teams. The bridge inspection inventory items are being scrutinized to determine if changes in the system are warranted. Computer checks are being made on the coding of bridge inspection items for quality control by cross checking various items including ratings and also for consistency with the original design specifications. The Safety Board believes that these changes will improve the NYSDOT bridge inspection program.

#### FHWA's Overview of NYSDOT's and NYSTA Bridge Inspection Programs

Since 1983, the Safety Board has investigated two other accidents involving collapses of major highway bridges -- the collapse of the bridge carrying Interstate 95 over the Mianus River in Greenwich, Connecticut, on June 28, 1983, and the collapse of the Chickasawbogue Bridge near Mobile, Alabama, on April 24, 1985.

In the Mianus River Bridge investigation, the Safety Board determined that the cause of the collapse was the lateral displacement of hangers by corrosion-induced forces due to deficiencies in the State of Connecticut's bridge safety inspection and bridge maintenance program. The probable cause determined in the Chickasawbogue Bridge collapse was the undetected deterioration of steel H-piles due to inadequate inspection of the underwater bridge elements by the State of Alabama. Contributing to this accident was the failure of the FHWA to adequately oversee the bridge inspection program of Alabama in accordance with the NBIS.

The Safety Board found in its investigation of the collapse of the Mianus River Bridge, and again in its investigation of the collapse of the Chickasawbogue Bridge, that the annual reviews of the State bridge inspection programs performed by the FHWA are essentially "paper audits." Reviews by the FHWA division, region, and headquarters do little more than verify that State bridge inspection reports -- the Structure Inventory and Appraisal Sheet (SI&A) -- are completed and that all the boxes are checked on the SI&A sheets. The sufficiency ratings developed from the SI&A sheets are used to establish priorities for rehabilitation or replacement projects. While the FHWA personnel do visit the field to observe bridge inspectors in action, the visits are not frequent enough for FHWA personnel to observe the inspection of all types of bridges within the States.

The need for proper audits and reviews was highlighted in both investigations. As a result of the Mianus River Bridge investigation, the Safety Board, on July 19, 1984, issued Safety Recommendation H-84-56 to the U.S. Department of Transportation (DOT):

Direct the DOT Inspector General to review the Federal Highway Administrator's bridge inspection audit program for its sufficiency in establishing State compliance with the National Bridge Inspection Standards.

The DOT responded that three Inspector General audits of FHWA bridge programs had been expanded to include bridge inspection, and that the Inspector General's office would review the effectiveness of FHWA in obtaining compliance with the NBIS. As a result of this action, recommendation H-84-56 has been classified "Closed--Acceptable Action."

On May 15, 1987, the DOT Office of Inspector General Region 3 forwarded copies of a final report on the audit of the Quality of Bridge Inspection to the FHWA Region 1 Administrator. The audit had been conducted at the FHWA Region 1 office, the FHWA division offices, and State Highway offices in three Region 1 States -- New York, New Jersey, and Vermont. The audit included a review of bridge inspections made predominantly from June 1984 to June 1986, and its objectives were to evaluate the adequacy of

(1) State Highway Agency Office bridge inspection procedures and internal controls for ensuring that bridge inspections were complete and thorough and (2) FHWA policies and procedures for determining whether the State Highway offices' bridge inspection programs were in compliance with the NBIS.

The results of this audit indicated that management of the bridge inspection programs in FHWA Region 1 needed improvement because full compliance with the NBIS in Region 1 had never been achieved during the preceding 15 years. Among other deficiencies, the audit indicated that before June 1985, the FHWA had not emphasized underwater inspections and had not required the FHWA divisions to review the States' underwater inspection capabilities. Following a June 26, 1985, FHWA directive on NBIS underwater inspections, the audit indicated that the three States had performed underwater inspections on selected bridges but had not established formal comprehensive programs to identify all bridges requiring an underwater inspection. In New York, only 2 of the 11 transportation regions in the State had performed any underwater inspections.

Although the FHWA audit report made no recommendations concerning underwater inspections, it concluded that the FHWA Region 1 bridge inspection program was below standard primarily because the region had not required the States to allocate sufficient resources to bridge inspection programs and to the development of capable inspection organizations. Further, Region 1 had not taken aggressive action such as the temporary suspension of Federal aid to encourage the States to comply with the NBIS.

FHWA data indicated that 5 to 6 percent of all bridges in New York State were overdue for inspection in 1986 and 1987. However, the FHWA was already aware that the NYSTA was not inspecting its bridges within the time specified by the NBIS. In its 1986 review of the New York State Bridge Program, the FHWA New York division office pointed out that of the approximately 250 bridges that the NYSTA needed to inspect, 50 percent had not been inspected within the last 2 years. In addition, the below-water substructural components of several of these bridges, including the Schoharie Creek Bridge, had never been inspected.

On January 27, 1988, the DOT Office of the Inspector General informed the Safety Board that it had completed an audit of the FHWA National Bridge Inspection Program (NBIP) for the period from January 1984 through June 1986. The objectives of the audit were to evaluate the adequacy of the States' programs for conducting bridge inspections and FHWA's controls for managing the NBIP. They found weaknesses in the bridge inspection programs of the seven States audited, including New York State. The audit, which included information from the Region 1 audit previously mentioned, showed that States had not performed underwater inspections, established adequate internal controls, or conducted thorough inspections. While FHWA has acted to strengthen its controls for managing the NBIP, the Office of the Inspector General found that the FHWA had not adequately (1)

monitored essential elements of the States' bridge inspection programs, (2) ensured that States were providing written responses indicating the corrective action taken on identified deficiencies, and (3) evaluated the FHWA divisions' monitoring of the States' bridge inspection programs. Further, the Inspector General found that these conditions existed because (1) standards and other criteria did not clearly require the State to perform underwater inspections and establish internal controls over bridge inspections, (2) States did not have the proper equipment available for making inspections, (3) States were not required to document corrective actions taken on deficiencies reported by bridge inspectors, (4) States had not allocated sufficient resources to the bridge inspection program, and (5) the FHWA had not established sufficient control for monitoring the States' bridge inspection programs. The DOT Inspector General also made eight recommendations. The FHWA's responses to the recommendations were published in the DOT Inspector General's report. Appendix H contains the recommendations and the FHWA's responses.

Based on its prior investigations of bridge accidents and on the DOT Inspector General's findings and recommendations, the Safety Board concludes that, as an agency, the FHWA has lacked aggressiveness and initiative in formulating and implementing a comprehensive bridge inspection program among the States. Moreover, the FHWA has been particularly slow to encourage the States to adopt comprehensive underwater inspection programs and to provide guidance on the proper inspection techniques and procedures that should be employed.

With regard to the State of New York, the Safety Board believes that despite the distinct institutional difference between NYSDOT and NYSTA, the FHWA should have held the State responsible for the inspection of all bridges on public roads, including the bridges on the Thruway, in accordance with the NBIS, and withheld Federal aid pending NYSDOT's acceptance of its responsibility. As the matter stood, NYSTA's inadequate inspections, although reported to NYSDOT, were never carefully scrutinized to detect and correct the inadequacies.

The Safety Board recognizes that FHWA management's response to the DOT Inspector General's recommendations, if fully implemented, will correct many of the deficiencies with the NBIP as implemented by the various States. However, the Safety Board also recognizes that the FHWA has had difficulty in the past in obtaining State compliance with the NBIS and with the development of programs to provide adequate guidance on inspection techniques and procedures. Consequently, the Safety Board believes that regular scrutiny by the DOT Inspector General is needed to ensure effective FHWA oversight of the States' compliance with the NBIS.

## Research into Scour and Riprap Stability

Adequacy of Scour Research.--Currently, several programs on scour research are underway both at the State and Federal level. The purpose of these programs is to evaluate the performance of bridge foundations during floods, to develop more reliable and accurate scour prediction equations, and to identify design changes that will improve bridge stability during floods. In addition, the FHWA plans to issue a technical advisory on scour during the summer of 1988. The Safety Board is encouraged by the Federal-State cooperative scour research program currently being implemented in several States. 70/ This cooperative effort can minimize the level of duplication among agencies and the time required to complete this important research. The Safety Board believes that because current methods for estimating stream velocities and predicting scour depth are more of an art than a science, continued research is needed to improve present methods for determining the potential for scour at selected bridge sites.

In this accident, New York State Police traveled over the bridge about 5 minutes before its collapse and did not notice anything unusual about the bridge or its riding surface. Since the State Police did not have means to determine the condition of the bridge or the danger that the flood imposed, the bridge was not closed.

Since it is unlikely that monitoring teams will be available at all bridges during flooding if and when devices such as truck mounted fathometers are fully developed, other warning systems need to be developed. The Safety Board believes that FHWA should perform research on simpler methods that could provide a warning of the extent of scour or the severity of flooding at bridges over water, especially for those supported by spread footings.

Alternatives suggested by some engineers include resistance type scour meters, a sleeve and pipe combination in which the pipe would sink into a developing scour hole, visually different layers of riprap material, and burial of tethered floats in the riprap. These types of devices could be installed during rehabilitation projects.

Perhaps some bridges could even be painted with a design highwater mark. When flood waters approach the highwater mark, a bridge engineer could be contacted for further evaluation or the police could close the bridge.

70/ Virginia, Maryland, Delaware, Arkansas, New York, Oregon, Ohio, Connecticut, Pennsylvania, and California.

At the Schoharie Creek Bridge, the anticipated extreme high water elevation was 290 feet. At the time of collapse, water was at an elevation of about 296 feet. As the water elevation increases, velocities generally increase as does scour depth. It may be prudent to close bridges built on spread footings when the water elevation has exceeded a level specified by the designer. The availability of simple warning devices may make this possible.

Adequacy of Riprap Research.--The Safety Board believes that not enough data is currently available to determine with sufficient accuracy and reliability the movement of riprap under different stream conditions. Current research can only establish a broad range of velocities that would move riprap of a given size (weight). Thus, riprap stability analyses can provide bridge designers with only very rough estimates of sizes of riprap needed to protect a bridge foundation not on piles. Present equations and calculations used to predict riprap stability need to be refined.

The variations in the data provided by MRCE and CSU on the magnitude of stream velocities needed to move riprap of given weight are an indication of the uncertainty that now exists in specifying the size of riprap needed for scour protection. (See table 7 and figure 37.) The median weight of the item 80 riprap at the bridge was to have been 300 pounds. CSU concluded that the conditions in a turbulent stilling basin best represented conditions at pier 3. CSU found that a 300-pound rock would move in water with a velocity of 7.9 fps. However, CSU also stated that this velocity could vary by as much as 25 percent, which indicates a 300-pound rock could move in water at a velocity of from 6 to 10 fps. MRCE data indicate that a 300-pound rock would move in water at 10.1 to 10.8 fps.

These data can be compared to the physical evidence at the accident site and to the results of the CSU modeling to gain a better perspective of the meaning of these data. Before 1979, riprap moved from around pier 2. The results of the three-dimensional physical modeling indicated that at flows between 60,000 and 73,600 cfs, the velocities at pier 2 were between 6.0 to 8.5 fps. According to CSU, this velocity was sufficient to move the riprap around pier 2. This suggests that the conditions selected by CSU may have been correct. However, according to the MRCE data, the riprap would not have moved from above the footing at these speeds. This observation would tend to indicate that the CSU data is a better predictor of riprap movement.

However, photographs also show movement of stone between 1956 and 1977 at pier 2 when floods were between 30,000 and 35,500 cfs and the expected velocity at this pier for these flows based on the CSU three-dimensional model was only 3 fps. This

low velocity would not have been sufficient to move the riprap at pier 2. This would suggest that the CSU data are not very good for predicting riprap movement at low speeds or that perhaps ice or some other phenomenon was responsible for the movement of the riprap at pier 2.

It can be shown that the weight of riprap necessary to resist movement by a streamflow varies to the sixth power of the velocity of the stream. Thus, small increases in the stream velocity can result in the movement of much larger riprap. Because this weight-velocity relationship is so sensitive to changes in velocity, the designer must specify extremely large riprap if fairly large velocities are predicted. Research is needed to refine the existing data and predictive methodologies. Until this research is completed, designers should not depend on riprap to protect spread footings.

The current AASHTO Manual for Bridge Maintenance - 1987 and its predecessor (1976) provide a "rough guide to the selection of an adequate stone size at bridge crossings." This table indicates that riprap with an average stone size of 600 pounds will provide adequate scour protection even when the average velocity of a stream is 10 to 15 fps. The results of the RCI/CSU study show that the velocity at a pier can be greater than the average stream velocity as a result of bends in the streambed. Further, the curve data for the turbulent stilling basin indicate that velocities of 10 to 15 fps could move riprap weighing as much as 1,000 to 6,000 pounds. The other AASHTO specifications of 6-inch stone for streams with velocities up to 7 fps and 100-pound stone for velocities of 7 to 10 fps also appear to be too low when compared to the movement of 300-pound riprap around piers 2 and 3 of the Schoharie Creek Bridge. The Safety Board believes that AASHTO should issue an addendum to this recent publication to caution the user not to rely on the information in the table.

Currently, the cooperative Federal-State research is focused on determining the variables that influence scour. The Safety Board believes that the scope of this research effort should be expanded to include the study of riprap stability.

#### Emergency Response and Emergency Preparedness

Emergency personnel arrived at the bridge site about 5 minutes after the collapse of the first two spans, established a command center, and began directing traffic control. Although a shoreline search for survivors was initiated, no survivors were found and no medical attention was required. Coordination among responding agencies was good and no deficiencies were noted.

Similarly, emergency preparedness did not affect the outcome of this accident. During the weekend of April 3 through April 5, 1987, Montgomery County officials worked very closely with officials from the New York State Emergency Management Office to maintain a high state of readiness for problems

associated with severe flooding along the Schoharie Creek basin. They activated the County Disaster Plan, which allowed county officials to monitor continuously the elevation of the creek, to alert and evacuate residents, and to close roads and bridges over the creek when water levels became dangerously high.

The actions taken by the State and County officials were timely, appropriate, and well coordinated. Had these officials waited until April 5 to take action, the extraordinary flooding along the Schoharie Creek basin could have resulted in a greater loss of life and property.

## CONCLUSIONS

### Findings

1. Neither the performance of the motorists nor their vehicles were factors in this accident.
2. The emergency response for this accident was effective for the accident conditions.
3. The actions taken by the State and County officials in preparation for the severe flooding along the Schoharie Creek basin were timely and well coordinated.
4. Local scour eroded the soil beneath pier 3 causing the pier to rupture, the upstream portion of the pier to drop into the scour hole, and spans 3 and 4 to collapse.
5. The bridge wreckage from the collapse of pier 3 redirected the water flow, causing rapid erosion of the soil under pier 2, the collapse of pier 2 into the scour hole, and the collapse of span 2.
6. The lowering of the streambed around pier 2 during construction and its subsequent buildup with fill material made the bed around pier 2 more susceptible to scour.
7. If scour piles had been used, they may have prevented the collapse of piers 2 and 3 depending on how deeply they were driven below the footings; however, AASHTO specifications and the design standards of the early 1950s did not clearly specify when piles should be used, and riprap was an acceptable alternate means of protection against scour. Without piles, the Schoharie Creek Bridge was completely dependent on the maintenance of riprap to protect its foundation against scour.

8. If steel sheeting used during the construction of the bridge piers had been left in place, it may have altered the collapse sequence for the bridge piers and extended somewhat the time until the collapse occurred; however, the shallow sheeting would not have prevented the collapse of pier 3.
9. As a result of ambiguity in the design plans and specifications, riprap was never placed to the bottom of the footings and this contributed to the collapse of the bridge.
10. A significant amount of the riprap placed around piers 2 and 3 at the time of construction moved northward along the piers during various floods that took place from 1955 to 1987, enabling the footings to be undermined during the flood of April 1987.
11. New York State Thruway Authority inspections of the Schoharie Creek Bridge did not document the elevation or condition of the streambed or underwater elements of the substructure.
12. Bridges with piers that have shallow spread footings and riprap for protection from scour require more frequent and more thorough inspections if the water has the potential to be turbulent than do bridges located in placid water.
13. Because of the shallow spread footings and the erodability of the soil during flood conditions, maintenance of sufficient riprap protection around the Schoharie Creek Bridge piers was paramount to ensuring the overall safety of the bridge.
14. The lack of structural redundancy in the design of the two-girder, simply supported, non-continuous span bridge contributed to the rapid and catastrophic nature of the collapse.
15. Structural members and foundation features critical to the integrity of the bridge were not highlighted in the design plans or recognized by the bridge inspectors.
16. Consultants for the New York State Thruway Authority who inspected the bridge in 1977 noted that riprap was missing around piers 2 and 3. Although the bridge rehabilitation plans prepared by the consultants specified the replacement of missing riprap with heavier stone, the New York State Thruway Authority deleted the specification for riprap from the contract.

17. A consultant who inspected the bridge for the New York State Department of Transportation in 1979 documented scour at piers 2 and 3; this information was not acted upon by the NYSDOT or by the New York State Thruway Authority.
18. Inspectors (and some supervisors) from the New York State Thruway Authority, the New York State Department of Transportation, and Seelye, Stevenson, Value, and Knecht either failed to understand the importance of riprap or failed to recognize that sufficient riprap had migrated from around piers 2 and 3 to pose a danger to the bridge.
19. The American Association of State Highway Officials and the New York State Department of Transportation should modify their guidance for the replacement of riprap to specify that following each inspection of a bridge that is dependent upon riprap for scour protection, any missing riprap must be replenished to design specifications.
20. If the New York State Thruway Authority had replaced the missing riprap with 600-pound stones as recommended in 1977 by the consultant engineering firm, the bridge probably would not have collapsed during the April 1987 flood.
21. If New York State specification item 80 riprap had been maintained in a manner and to a level as originally placed around the piers, the bridge probably would not have collapsed.
22. The FHWA should require States to locate and inspect all two-girder bridges that are simply supported, have non-continuous spans, and are built with shallow spread footings in streambeds to determine the condition of their foundations.
23. Regular scrutiny by the U.S. Department of Transportation Inspector General is needed to ensure effective FHWA oversight of the States' compliance with the National Bridge Inspection Standards.
24. The guidance on stone size provided in a chart on page 160 of the AASHTO Manual for Bridge Maintenance - 1987 probably underestimates, significantly, the size of riprap that will protect against floods of given velocities.
25. The scope of the Federal-State cooperative research program should be expanded to include the documentation and analysis of riprap stability at applicable bridge sites.

### Probable Cause

The National Transportation Safety Board determines that the probable cause of the collapse of the Schoharie Creek Bridge was the failure of the New York State Thruway Authority to maintain adequate riprap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the accident were ambiguous plans and specifications used for construction of the bridge, an inadequate NYSTA bridge inspection program, and inadequate oversight by the New York State Department of Transportation and the Federal Highway Administration. Contributing to the severity of the accident was the lack of structural redundancy in the bridge.

### RECOMMENDATIONS

As a result of its investigation, the National Transportation Safety Board recommends:

--to the American Association of State Highway and Transportation Officials:

Revise section 4.4.2.1 of the Standard Specifications for Highway Bridges removing any reference to a minimum depth of 4 to 6 feet for bridges over water and stating instead that the minimum depth of footing be based on historical data for scour at or near the bridge site and mathematical analyses of maximum potential scour depth. (Class II, Priority Action) (H-88-12)

Modify section 4.3.1.2 of the Standard Specifications for Highway Bridges to require that the depth of piles exceed the predicted maximum potential depth of scour. (Class II, Priority Action) (H-88-13)

Provide specific guidance on maintenance of foundations of bridges that are dependent upon riprap for scour protection to specify that following each inspection the riprap be replenished to meet the design specifications. (Class II, Priority Action) (H-88-14)

Issue an addendum to the Manual for Bridge Maintenance-1987 that strongly cautions the user that the chart on page 160, which is a "rough guide to the selection of an adequate stone size at bridge crossings," is probably inadequate to ensure that all the riprap will remain in place under the average velocities listed; issue an errata sheet to correct previous editions. (Class II, Priority Action) (H-88-15)

--to the Federal Highway Administration

Expand the scope of the Federal-State cooperative research program on this evaluation of bridge performance during flood conditions to include the documentation and analysis of riprap stability at applicable bridge sites; disseminate the results of this expanded study to owners of bridges. (Class II, Priority Action) (H-88-16)

Encourage States to conduct in-depth hydraulic studies during rehabilitation and reconstruction of bridges over water to determine if changes in the stream flow and streambed have affected the adequacy of the initial design. (Class II, Priority Action) (H-88-17).

Require all States that have not done so within the last year to conduct underwater inspections of all their bridges founded on spread footings, for evidence of scour (such as movement of riprap, development of scour holes in the streambed, and changes in the streambed material composition), placing priority on those bridges with nonredundant designs. (Class I, Urgent Action) (H-88-18)

Compel, by withholding Federal funds if necessary, the owners of all public bridges including those not owned by the States, to comply with the National Bridge Inspection Standards. (Class II, Priority Action) (H-88-19)

Research methods by which alerting signs and detection devices could be placed on or near bridges for observation during flood conditions to aid in the decision to close bridges, particularly those built on spread footings. (Class II, Priority Action) (H-88-20)

--to U.S. Department of Transportation:

Direct the DOT Inspector General to periodically review the Federal Highway Administration's bridge inspection audit program for its sufficiency in establishing State compliance with the National Bridge Inspection Standards. (Class II, Priority Action) (H-88-21)

--to New York State Department of Transportation:

Modify the guidance provided in the New York State Department of Transportation Highway Maintenance Guidelines to require that, following each inspection of bridges that are dependent upon riprap for scour protection, the riprap be replenished to meet the design specifications, and remove the sentence in

Section 4.4.5.2 of the guidelines, which states, "Repairs should be made, using heavy stone fill or riprap before scour progresses to a depth dangerous to the stability of a structure (1/2 of the thickness of pier footing)." (Class II, Priority Action) (H-88-22)

--to the American Association of State Highway and Transportation Officials, the International Bridge, Tunnel and Turnpike Association, the National Association of Counties, the National League of Cities, and the National Association of Towns and Townships:

Inform your members of the circumstances of the Schoharie Creek Bridge collapse of April 5, 1987, and alert them of the importance of the inspection and maintenance of riprap at bridges founded on spread footings that rely upon the riprap to protect the spread footings from scour. (Class II, Priority Action) (H-88-23)

As a result of its investigation of this accident, the National Transportation Safety Board reiterates Safety Recommendation H-86-3 to the Federal Highway Administration:

Establish criteria for inspecting the underwater elements of bridges which consider the following facts as they relate to bridge design and maintenance:

Complexity of structure and materials used,  
Marine environment surrounding the underwater  
elements of the bridges, and  
Frequency and magnitude of loads on the bridge.

BY THE NATIONAL TRANSPORTATION SAFETY BOARD.

/s/ JIM BURNETT  
Chairman

/s/ JAMES L. KOLSTAD  
Vice Chairman

/s/ JOHN K. LAUBER  
Member

/s/ JOSEPH T. NALL  
Member

April 29, 1988

**APPENDIX A**  
**INVESTIGATION AND HEARING**

**Investigation**

The National Transportation Safety Board received notification of the accident on April 5, 1987. The investigation was organized into investigative groups led by Safety Board personnel. Group members represented the State of New York, the Federal Highway Administration, the National Weather Service, and the U.S. Geological Survey.

The Safety Board worked very closely on this investigation, with the State of New York and several consultants retained by the State. Information concerning the examination of bridge wreckage, structural evaluation of primary bridge elements, and geotechnical examination and analysis of the streambed and bridge site was developed by Wiss, Janey, Elstner Associates, Inc., (WJE) of Northbrook, Illinois, and Mueser Rutledge Consulting Engineers (MRCE), New York, New York.

Extensive surveys, studies, and tests were conducted. Surveys were made to document changes to the streambed profile. Tests were conducted to determine the bearing capacity and erodability characteristics of the soil at the bridge site. Tests were performed to quantify the concrete strength and span stiffness of bridge members. A finite element analysis of the pier 3 substructure was performed. All information developed was shared with the Safety Board staff as it became available.

With the State of New York, the Safety Board also commissioned a study to determine how erosion, scour, and hydraulic forces may have affected the collapse of the bridge. Resource Consultants, Inc. (RCI) and the Colorado State University (CSU), both in Fort Collins, Colorado, were contracted to perform the hydraulic study.

RCI conducted an analytical study to evaluate the influence of geomorphic, hydrologic, and hydraulic conditions on flood routing and attenuation along the Schoharie Creek basin through mathematical computer modeling techniques. CSU performed flume studies of two- and three-dimensional physical models of the stream profile and surrounding terrain at the bridge site to obtain additional parameters to empirically quantify the extent of scour and channel instability. The results of the RCI study have been incorporated into the text of the report.

The two final reports concerning the collapse of the Schoharie Creek Bridge have been published by consultants hired by the State of New York. The reports are entitled "Collapse of the Thruway Bridge at Schoharie Creek for New York State Thruway Authority, Albany, New York" prepared by WJE and MRCE, November,

1987, and the "Overview Report Investigation of the New York State Thruway Schoharie Creek Bridge Collapse" prepared by Thorton-Tomasetti (P.C.), December 1987.

### Hearing

The Safety Board convened a public hearing in Albany, New York from June 30 to July 2, 1987, to inquire further into the bridge collapse. The Safety Board examined several factors surrounding the investigation, specifically the adequacy of the designed, maintenance, and inspection of this bridge, and the adequacy of State and Federal oversight of bridge management programs.

## APPENDIX B

## VEHICLE AND OCCUPANT INFORMATION

A. 1984 PLYMOUTH GRAN FURY, BLUE, 4 DOOR, VIN  
IP3BB26PSEX536620

1. Mary H. Peck, 47, of Schylerville, New York, was the driver of the vehicle. She had a valid New York state operators permit.
2. Kristen Peck, 22, of Schylerville, New York, was a passenger in the vehicle.

The cause of death for both occupants was "Multiple Fractures and Internal Injuries". Both occupants were found in the vehicle on April 11, 1987.

B. 1987 CADILLAC, SEDAN de VILLE, GREY, 4 DOOR, VIN  
IGGCD698474394622

1. Jackson C. Dalton, 65, of Mississauga, Ontario, Canada, the driver of the vehicle, had a valid Ontario, Canada, operators permit.
2. Roland Charbonneau, 61, of Toronto, Ontario, was a passenger in the vehicle.

Both occupants were found in the vehicle on April 6, 1987.

C. 1981 CADILLAC, FLEETWOOD, WHITE, 4 DOOR, VIN  
IG6AB6987C9136915

1. Douglas Shive, 68, of Manchester, New Hampshire, the driver, was found in the vehicle on April 11, 1987.
2. Evangeline Shive, 71, of Manchester, New Hampshire. She was the passenger in the vehicle. Her body was recovered April 22, 1987, from the Hudson River in the town of New Baltimore, New York.

D. 1985 MERCURY TOPAZ, WHITE, 4 DOOR, VIN 2MEBP76X6FB608581

It is not known who was driving the automobile in which three males were traveling.

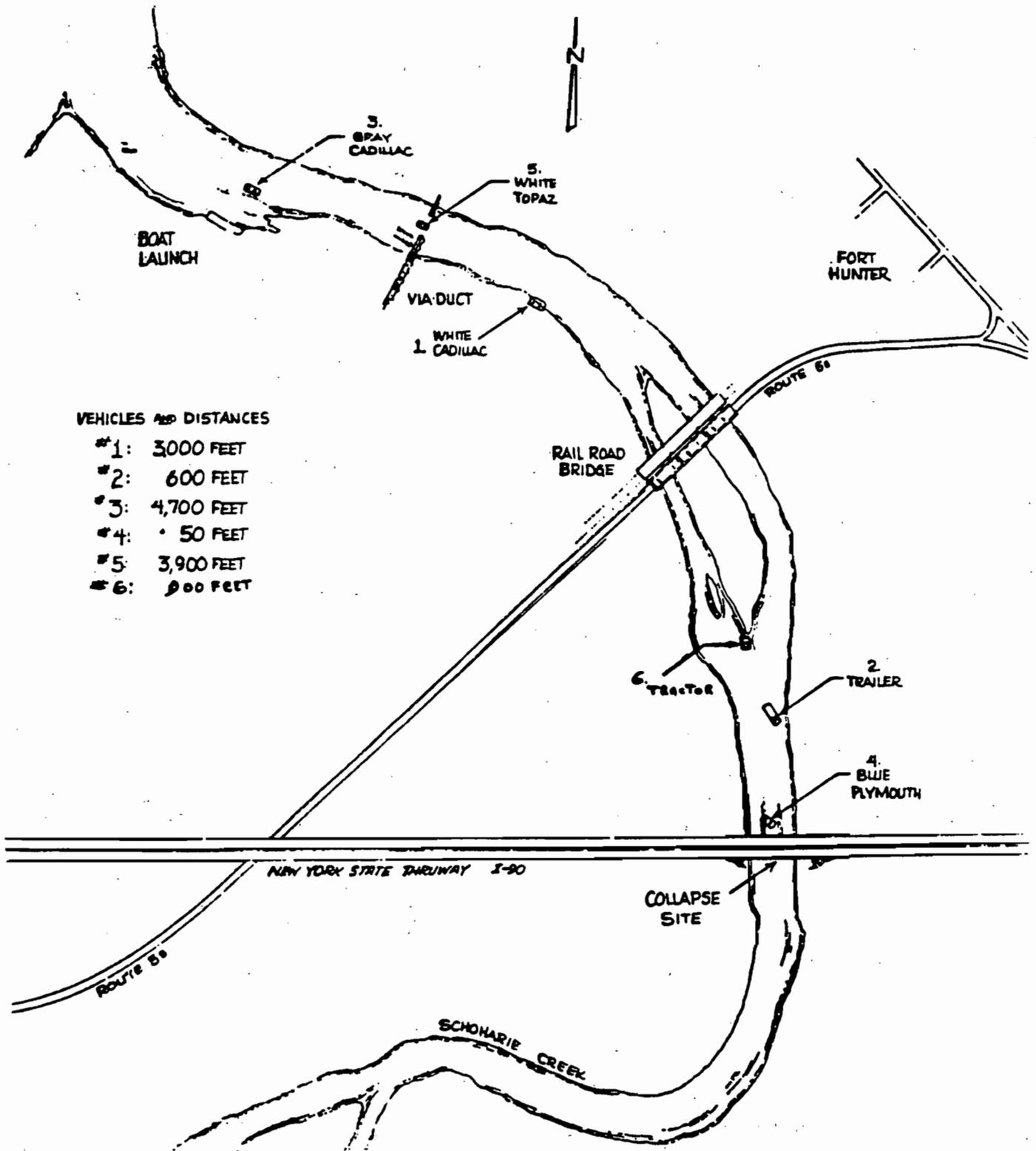
1. Donald Huges, 59, of Troy, New York. He was found in the vehicle on April 10, 1987.
2. Robert Hoffman, 46, of Rensselaer, New York. He was recovered from the Mohawk river in Alplause, New York, on April 26, 1987.

3. Edward Meyer, 50, of Albany, New York. He is still missing as of the date of this report.

E. 1982 INTERNATIONAL TRACTOR, VIN 1HT23270CG1108, 1985  
FRUEHAUF TRAILER, VIN FE007452

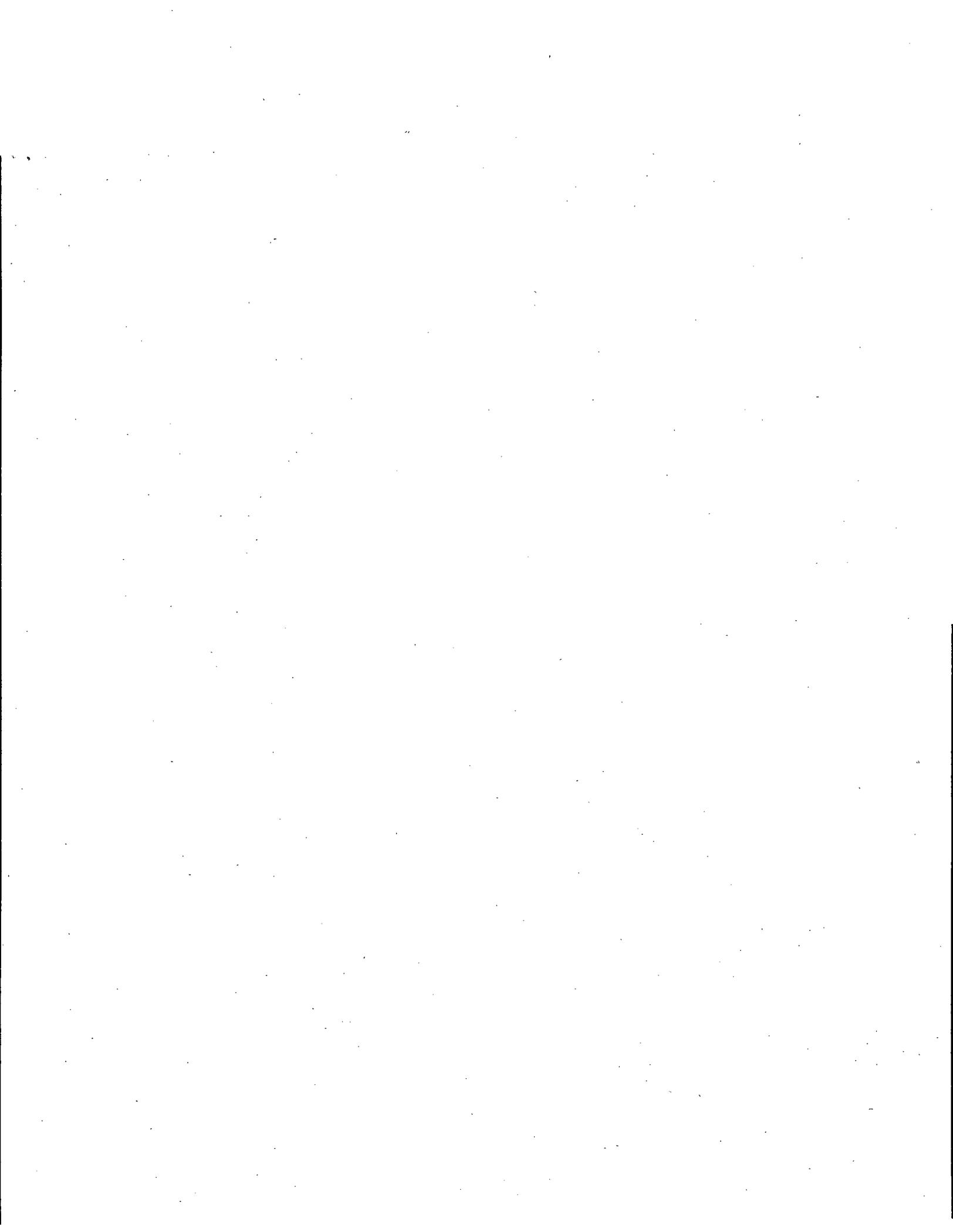
1. John Ninham, 39, of Greenbay, Wisconsin, had a valid Wisconsin commercial driver's license. He was recovered on April 22, 1987, in the tractor with his seat belt on.

# LOCATION OF VEHICLES AFTER ACCIDENT



**VEHICLES AND DISTANCES**

- #1: 3000 FEET
- #2: 600 FEET
- #3: 4,700 FEET
- #4: 50 FEET
- #5: 3,900 FEET
- #6: 900 FEET



## APPENDIX C

STATE EMERGENCY MANAGEMENT OFFICE (SEMO)  
NOTIFICATIONS AND ACTIVITIES ON APRIL 5, 1987

0110 Hour - SEMO Eastern District advised SEMO Chief of Staff that the Schoharie Creek had crested at Prattsville and Middleburg and that the County reports there have been no reports of fatalities or injuries. Implications are that waters should now begin to recede slowly.

0705 Hour - SEMO received request from Montgomery County for an air rescue operation of five stranded homeowners in Lost Valley, New York. This request was coordinated with the Division of State Police. They provided a helicopter for that purpose.

0900 Hour - Schoharie County reports to SEMO that water on Schoharie Creek is receding slowly and remains high. The County also reports that numerous county roads remain closed and a few bridges may be damaged. Damage assessments cannot be attempted until the water recedes. The County reports that the situation is improving since heavy rains have ceased. SEMO Chief of Staff provides field staff and Montgomery County briefing on this report from Schoharie County.

0932 Hour - SEMO Chief of Staff contacted the NWS in Albany for a weather forecast and is advised that the expected additional rains will cause problems in the Catskills but that the remainder of the State is not in any immediate danger as the situation is improving.

1015 Hour - SEMO provided U.S. Army Corps of Engineer representative with an update on statewide situation.

1020 Hour - SEMO Chief of Staff briefs the SEMO Director on situation and outlook.

1025 Hour - SEMO received an update from Schoharie County Emergency Manager indicating that damage assessment procedures for roads and bridges have been formulated and that recovery efforts are underway.

1035 Hour - State DOT Disaster Preparedness Commission (DPC) Liaison briefed on the potential road and bridge problems that are being reported by the counties and is advised that SEMO field staff will be contacting State Agency Regional Response Team members today to commence damage assessment on Monday morning.

1050 Hour - SEMO is advised by Montgomery County that a bridge on the Thruway, west of the city of Amsterdam, is reported to have collapsed. A tractor trailer and four automobiles may have been involved in the incident.

1052 Hour - SEMO reports Thruway bridge collapse to the DSP Communications Section and requested immediate response for on-site inspection and confirmation. SEMO also requested DSP to notify State Thruway Authority representative advising of the need for their response.

1120 Hour - SEMO received request from Montgomery County for State resources to assist County Sheriff in traffic control.

1122 Hour - SEMO contacted DSP and requested support for traffic control and to coordinate such resources with Montgomery County Sheriff.

1135 Hour - SEMO advised SEMO Region 2 Director to contact State Department of Transportation Regional Director and request that all bridges along the Schoharie Creek be surveyed immediately to determine integrity.

1224 Hour - SEMO Region 2 Director advises that DOT is surveying bridges and has now barricaded Route 5S.

The State Emergency Management Office (SEMO) maintained contact with the State Police during the subsequent rescue activities and did supply a 10KW generator for State Police use at the scene as well as the Mobile Command Vehicle.

## APPENDIX D

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS  
STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES  
1949

## 2.1.2.--Preservation of Channel.

Unless otherwise specified, no excavation shall be made outside of caissons, cribs, cofferdams, steel piling or sheeting, and the natural streambed adjacent to the structure shall not be disturbed without permission from the engineer.

## 2.1.3.--Depths of Footings.

The elevation of the bottoms of footings, as shown on the plans, shall be considered as approximate only and the engineer may order, in writing, such changes in dimensions or elevation of footings as may be necessary to secure a satisfactory foundation.

## 2.1.4.--Preparation of Foundations for Footings.

All rock or other hard foundation material shall be freed from all loose material, cleaned and cut to a firm surface, either level, stepped, or roughened, as may be directed by the engineer. All seams shall be cleaned out and filled with concrete, mortar or grout.

When masonry is to rest on an excavated surface other than rock, special care shall be taken not to disturb the bottom of the excavation and the final removal of the foundation material to grade shall not be made until just before the masonry is to be placed.

## 2.1.8.--Back-fill.

All material used for back-fill shall be of a quality acceptable to the engineer and shall be free from large or frozen lumps, wood, or other extraneous material.

All spaces excavated and not occupied by abutments, piers, or other permanent work shall be refilled with earth up to the surface of the surrounding ground, with a sufficient allowance for settlement. All back-fill shall be thoroughly compacted and, in general, its top surface shall be neatly graded.

### 2.15.6.--Stone Riprap for Foundation Protection.

Stone riprap for pier and abutment protection shall range, in size, up to derrick stone and shall be graded from coarse to fine in such manner as to produce a minimum of voids. It shall be deposited where directed; stone deposited contrary to directions will be considered wasted and will not be paid for.

### 3.1.1.--Determination of Waterway Area

For the determination of the waterway area to be provided by any drainage structure, a careful study shall be made of local conditions, including flood height, flow and frequency, size and performance of other openings in the vicinity carrying the same stream, characteristics of the change and of the watershed area, climatic conditions, available rainfall records and any other information pertinent to the problem and likely to affect the safety or economy of the structure.

In general, the waterway provided shall be sufficient to insure the discharge of flood waters without undue backwater head and at a velocity which will not increase the erosive action of the stream to such an extent as to endanger the structure.

### 3.1.2.--Restricted Waterways.

When it is necessary to restrict the waterway to such an extent that the stream will be discharged at erosive velocities, protection against damage due to scour shall be afforded by deep foundations, curtain or cut-off walls, riprap, streambed paving, bearing piles, sheet piles, or other suitable means. Likewise, embankment slopes adjacent to all structures subject to erosion shall be adequately protected by riprap, brush mattresses, tree retards, wing dams, or other suitable construction.

### 3.1.4.--Pier Spacing and Location

Piers shall be located in such manner as to meet the above specified requirements (i.e., paragraphs 3.1.1 through 3.1.3) for channel openings. They shall be located so as to afford the minimum restriction of the waterway, especially in the main stream channel. In general, piers shall be placed as nearly parallel with the direction of the stream current as is practicable, due consideration being given to the velocity and direction of current as both ordinary and highwater stages, so as to avoid such deflections of the current as might prove destructive as to foundations of the structure or to the adjacent stream banks.

### 3.5.1.--Piles

#### (a) General.

In general, piling shall be used when footing cannot, at a reasonable expense, be founded on rock or other solid foundation conditions permit the driving of piles they preferably, shall be used as a protection against scour, even though the safe bearing resistance of the natural soil is sufficient to support the structure without piling.

In general, the penetration for any pile shall be not less than 10 feet in hard material and not less than  $1/3$  the length of the pile nor less than 20 feet in soft material.

For foundation work, no piling shall be used to penetrate a very soft upper stratum overlying a hard stratum unless the piles penetrate the hard material a sufficient distance to rigidly fix the ends.

### 3.5.2.--Footings

#### (a) Depth.

The depths of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, the footings of all structures, other than culverts, which are exposed to the erosive action of stream currents, preferably shall be founded at a depth of not less than 4 feet below the permanent bed of the stream. Stream piers and arch abutments, preferably, shall be founded at a depth of not less than 6 feet below stream bed. The above preferred minimum depths shall increase as conditions may require.

NEW YORK STATE SPECIFICATIONS  
ITEM 78-STONE FILLING

- a. Work. Under this item the Contractor shall furnish and place acceptable stone in fills or cribs as shown on the plans or as ordered by the Engineer.
- b. Material. The material used shall be durable field or quarry stone. All stones shall be so placed as to make fill or crib of maximum stability.
- c. Measurement and Payment. The quantity of stone filling to be paid for under this item will be the number of cubic yards measured in its final position. The unit price bid shall include the cost of furnishing all labor, materials and equipment necessary to complete the work, except that any necessary excavation will be paid for under an appropriate item.

ITEM 80--RIPRAP

- a. Work. Under this item the Contractor shall furnish and place dry riprap as shown on the plans or as ordered by the Engineer.
- b. Material. Dry riprap shall consist of durable field or quarry stone each shaped as nearly as practicable in the form of a right rectangular prism. At least fifty percent of the stones shall weigh in excess of three hundred pounds each, and the remainder of the stones shall weigh from 100 to 300 pounds each. One dimension of each of the stones furnished shall be the thickness of the riprap as shown on the plans, and the stones shall be so laid that this dimension is perpendicular to the prepared bed.
- c. Method. The stones shall be placed so that the weight of the stone is carried by the underlying material and not be the adjacent stones. On slopes, the largest stones shall be placed at the bottom. All dry riprap shall be properly aligned and in close contact and shall rest on a 6-inch bed of stone chips, crushed stone, crushed slag or gravel. The spaces between the stones shall be filled with spalls of suitable size.
- d. Measurement and Payment. The quantity of dry riprap paid for under this item will be the number of cubic yards measure in the final position. The porous bed placed under the dry riprap will be included in the quantity of dry riprap and will be paid for as such.

ITEM 119--RUN OF BANK GRAVEL FILL

- a. Work. Under this item the Contractor shall furnish and place a run of bank gravel fill as shown on the plans or as ordered by the Engineer.

b. Material and Method. Run of bank gravel fill shall be a well graded material generally conforming to the specifications for Item 39G, except that by weight 100% shall pass a 4 inch square sieve, 30% to 65% shall pass a No. 4 mesh sieve and not more than 10% shall pass a No. 22 mesh sieve.

c. Measurement and Payment. The quantity of run of bank gravel fill to be paid for under this item will be the number of cubic yards measured in place between the maximum payment lines as shown or indicate on the plans.

#### ITEM 82--COFFERDAMS

a. Work. Under this item the Contractor shall furnish, place, maintain, and remove cofferdams and pumping equipment at the location indicated on the plans or called for in the proposal in order that work may be progressed as ordered by the Deputy Chief Engineer (Bridges).

b. Method. Cofferdams shall be constructed so as to keep the excavations free from water, ice or snow and shall be so constructed as to permit the excavations to be carried to depths up to 3 feet below the foundation elevations shown on the plans. Any and all damage caused by the failure of a cofferdam from any cause whatsoever shall be the responsibility of the Contractor. It shall also be his responsibility to protect any and all stream banks from erosion by reason of restriction of the channel caused by the erection of the cofferdams. Any material which erodes from the banks during the time that the cofferdams are in place shall be replaced by the Contractor at his own expense.

c. Materials. Cofferdams may be constructed of earth, earth-filled bags, sheet piling or any other materials which the Contractor may elect to use and which will allow the foundations to be placed with the cofferdams in an unwatered condition. Pumping equipment and bracing shall be of adequate quantity and capacity and shall be so arranged as to permit their proper functioning in connection with the cofferdams.

d. Measurement and Payment. The quantity of cofferdams to be paid for under this item will be the number of square feet measured as follows: the number of square feet of cofferdams to be paid for will be determined by multiplying the distance, from the elevation 3 feet higher than the highest water elevation recorded at the site during the period in which the cofferdams are in use, by measuring along the driving line of the sheeting in the case of sheet pile cofferdams and along the gravity axis of the section in the case of earth or earth-filled bag cofferdams. The unit price bid for this item shall include the cost of furnishing all labor, materials and equipment necessary to satisfactorily complete the work.

## ITEM 83ST--TEMPORARY STEEL SHEET PILING

- a. Work. Under this item the Contractor shall furnish, place, maintain, and remove temporary corrugated metal, timber or steel sheet piling and necessary walling and bracing, all of the type, and at the locations shown on the plans or when and as ordered in writing by the Engineer.
- c. Material. Temporary Corrugated Metal or Steel Sheet Piling. Temporary Corrugated Metal or Steel Piling need not be new, but the section modulus and the web thickness of the piling used shall be not less than that shown on the plans.
- d. Method. Steam or pneumatic hammers shall be used when piling is driven. Any material which stops the driving of sheet piling shall be removed by the Contractor. Payment for the removal of such material will be made under an appropriate item. Unless otherwise shown on the plans, upon completion of the structure, the Contractor may, at his option, remove the corrugated metal or steel sheet piling placed under this item, or leave the same in place after cutting off the tops at the elevation ordered by the Engineer.
- e. Measure and Payment. The quantity of temporary piling to be paid for under this item will be the number of square feet of piling placed in its planned or ordered position. The unit price bid for this item shall include the cost of furnishing all labor, material and equipment necessary to complete the work.

## Appendix E

**NATIONAL BRIDGE INSPECTION STANDARDS**  
**23 CFR PART 650**  
**SUBPART C**

**23 CFR Ch. I (4-1-85 Edition)**

**Subpart C—National Bridge  
 Inspection Standards**

Source: 36 FR 7851, Apr. 27, 1971, unless otherwise noted. Redesignated at 39 FR 10430, Mar. 20, 1974.

**§ 650.301 Application of standards.**

The National Bridge Inspection Standards in this part apply to all structures defined as bridges located on all public roads. In accordance with the AASHTO (American Association of State Highway and Transportation Officials) Highway Definitions Manual, a "bridge" is defined as a structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

(23 U.S.C. 144, 116(d), 318; 49 U.S.C. 1655; 23 CFR 1.48(b))

(44 FR 25435, May 1, 1979)

**§ 650.303 Inspection procedures.**

(a) Each highway department shall include a bridge inspection organization capable of performing inspections, preparing reports, and determining ratings in accordance with the provisions of the AASHTO Manual<sup>1</sup> and the Standards contained herein.

(b) Bridge inspectors shall meet the minimum qualifications stated in § 650.307.

(c) Each structure required to be inspected under the Standards shall be rated as to its safe load carrying capacity in accordance with section 4 of the AASHTO Manual. If it is determined under this rating procedure that the maximum legal load under State law exceeds the load permitted under the Operating Rating, the bridge must be posted in conformity with the AASHTO Manual or in accordance with State law.

(d) Inspection records and bridge inventories shall be prepared and maintained in accordance with the Standards.

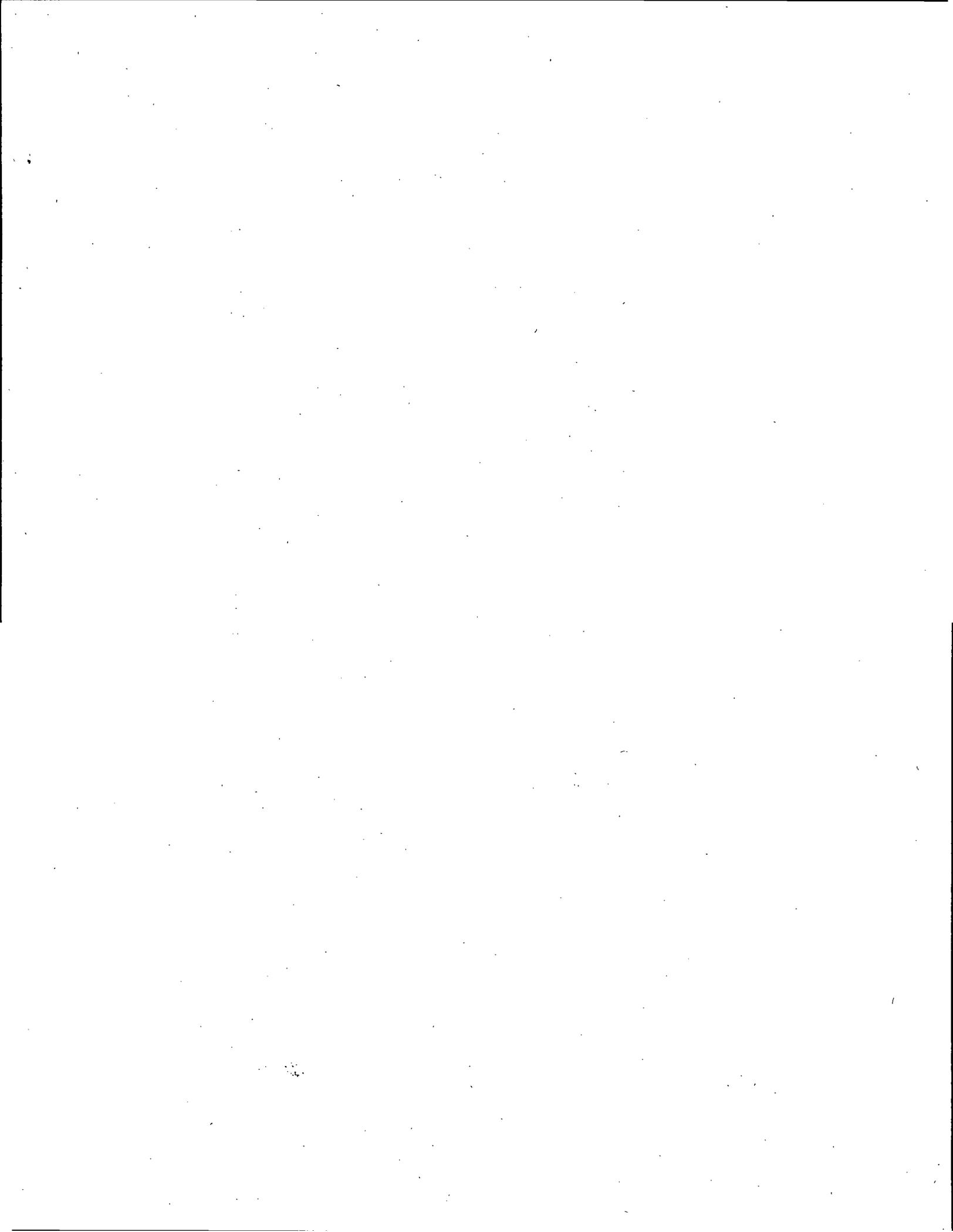
(23 U.S.C. 144, 116(d), 318; 49 U.S.C. 1655; 23 CFR 1.48(b))

(36 FR 7851, Apr. 27, 1971. Redesignated at 39 FR 10430, Mar. 20, 1974, and amended at 44 FR 25435, May 1, 1979)

**§ 650.305 Frequency of inspections.**

(a) Each bridge is to be inspected at regular intervals not to exceed 2 years

<sup>1</sup>The "AASHTO Manual" referred to in this part is the "Manual for Maintenance Inspection of Bridges 1978" published by the American Association of State Highway and Transportation Officials. A copy of the Manual may be examined during normal business hours at the office of each Division Administrator of the Federal Highway Administration, at the office of each Regional Federal Highway Administrator, and at the Washington Headquarters of the Federal Highway Administration. The addresses of those document inspection facilities are set forth in Appendix D to Part 7 of the regulations of the Office of the Secretary (49 CFR Part 7). In addition, a copy of the Manual may be secured upon payment in advance by writing to the American Association of State Highway and Transportation Officials, 444 N. Capitol Street, NW., Suite 225, Washington, D.C. 20001.



APPENDIX F  
RESULTS OF CSU PHYSICAL MODEL

TEST NO.	FLOW (CFS)	TAILWATER (FT-ELEV)	VELOCITY PROFILE	DEBRIS
1	62,100	294	yes	no
2	62,100	296	yes	no
3	62,100	292	yes	no
4	62,100	294	yes	yes
5	60,000	294	no	no
6	60,000	296	no	no
7	60,000	292	no	no
8	63,500	294	yes	no
9	63,500	295	yes	no
10	65,000	297	yes	no
11	73,600	296	yes	no
12	30,000	290	yes	no

TABLE 6.2. The test matrix for test runs on the 1:50 scale model, Schoharie Creek Bridge.

### SCHOHARIE CREEK VELOCITY CROSS SECTION

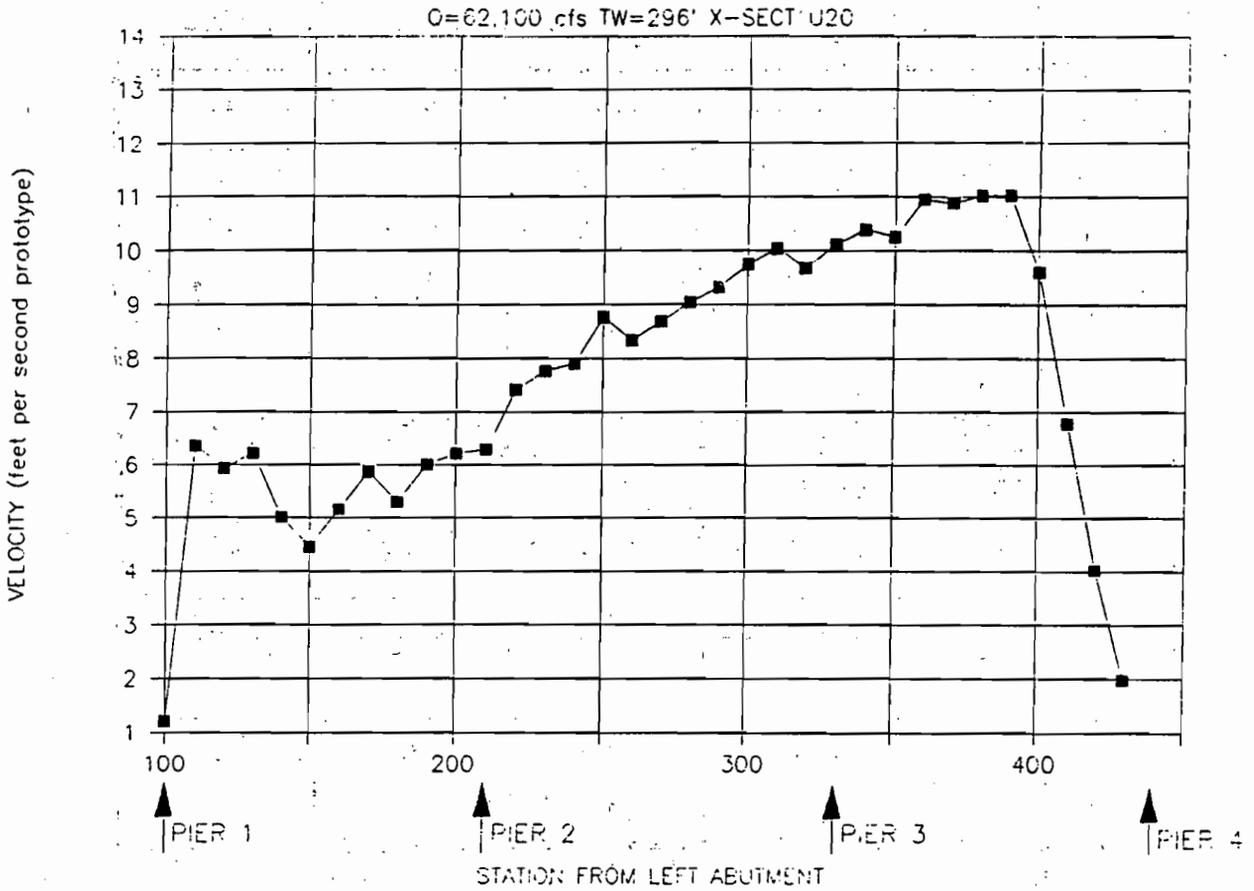


FIGURE 6.8. Velocity profile for the flood of 1987 taken from the upstream edge of the Schoharie Creek Bridge, with two foot higher tailwater than 1987 flood.

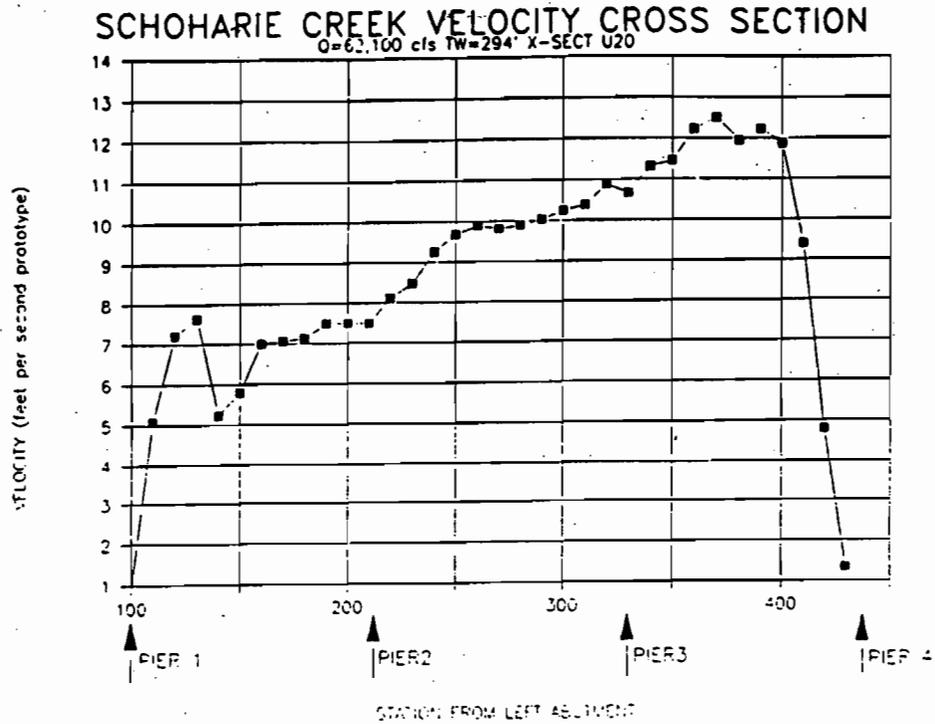


FIGURE D.2. Velocity profile for the flood of 1987 taken from the upstream edge of the Schoharie Creek Bridge.

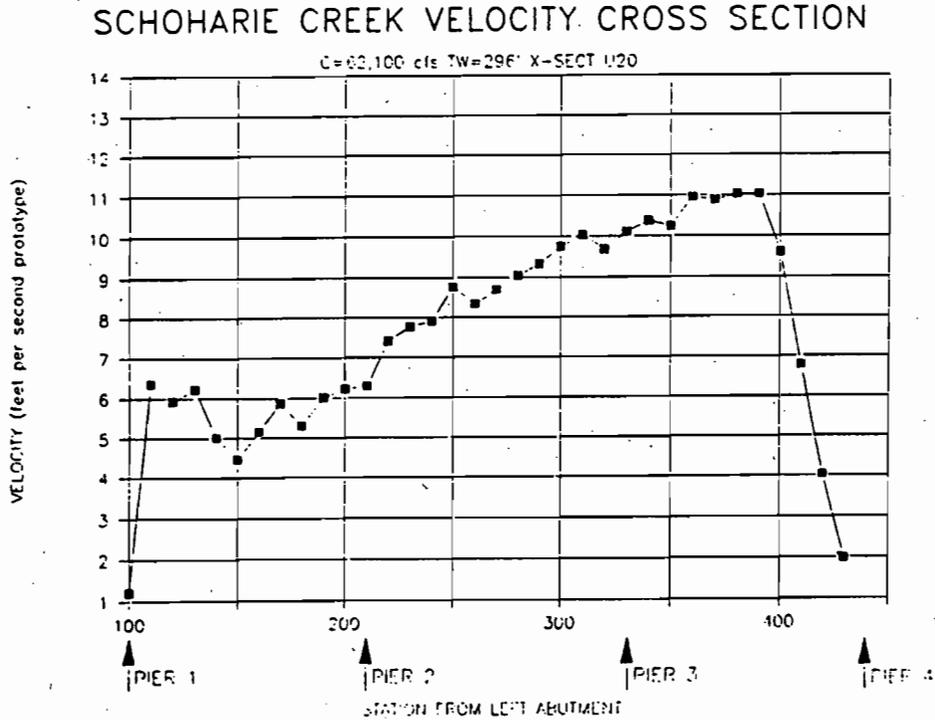


FIGURE D.3. Velocity profile for the flood of 1987 taken from the upstream edge of the Schoharie Creek Bridge, with two foot higher tailwater than the 1987 flood.

SCHOHARIE CREEK VELOCITY CROSS SECTION

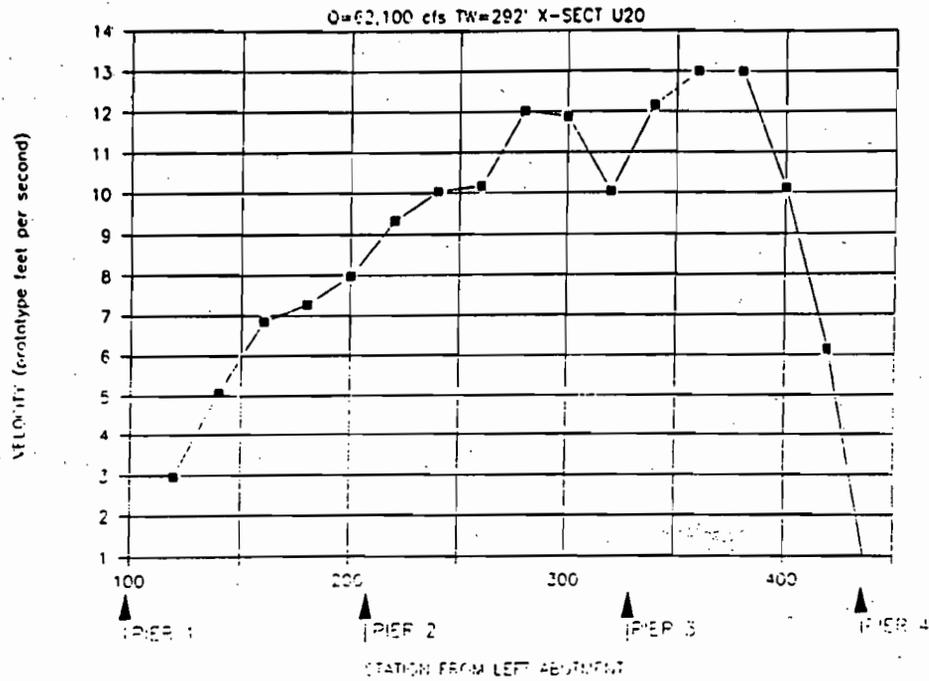


FIGURE D.4 Velocity profile for the flood of 1987 with two foot lower tailwater than the 1987 flood.

SCHOHARIE CREEK VELOCITY CROSS SECTION

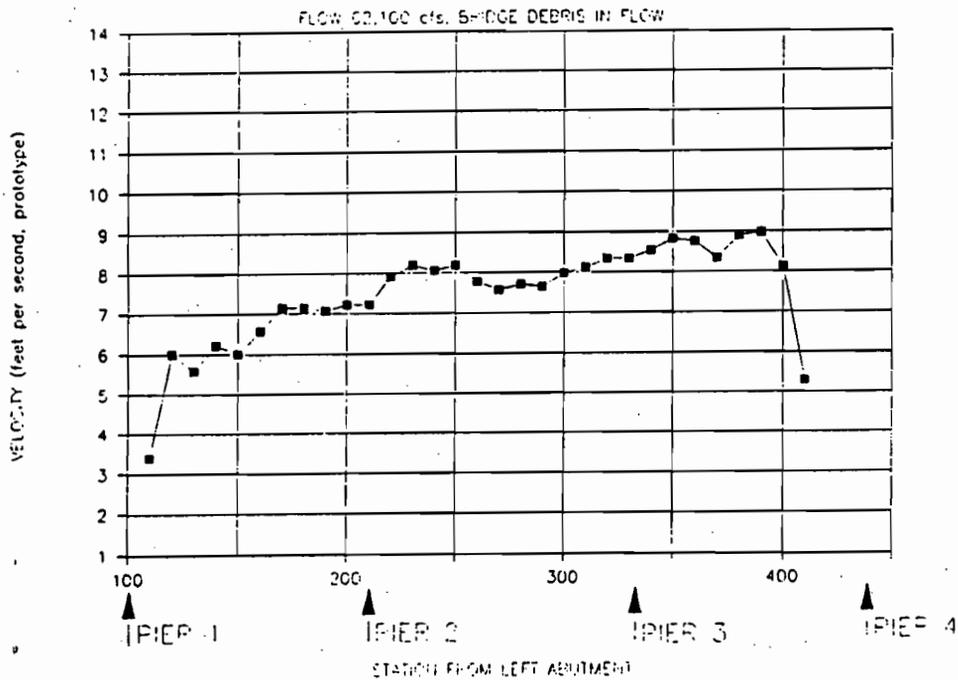


FIGURE D.5. Velocity profile for the flood of 1987 with bridge spans 3 and 4 in the water.

SCHOHARIE CREEK VELOCITY CROSS SECTION

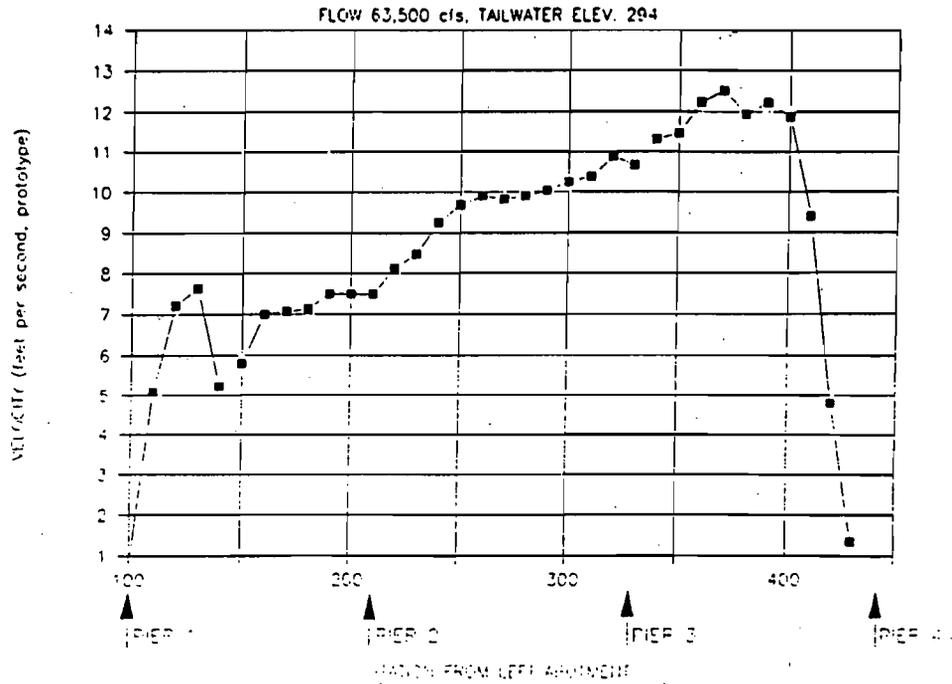


FIGURE D.6 Velocity profile for 63,500 cfs and a 294 feet elevation tailwater.

SCHOHARIE CREEK VELOCITY CROSS SECTION

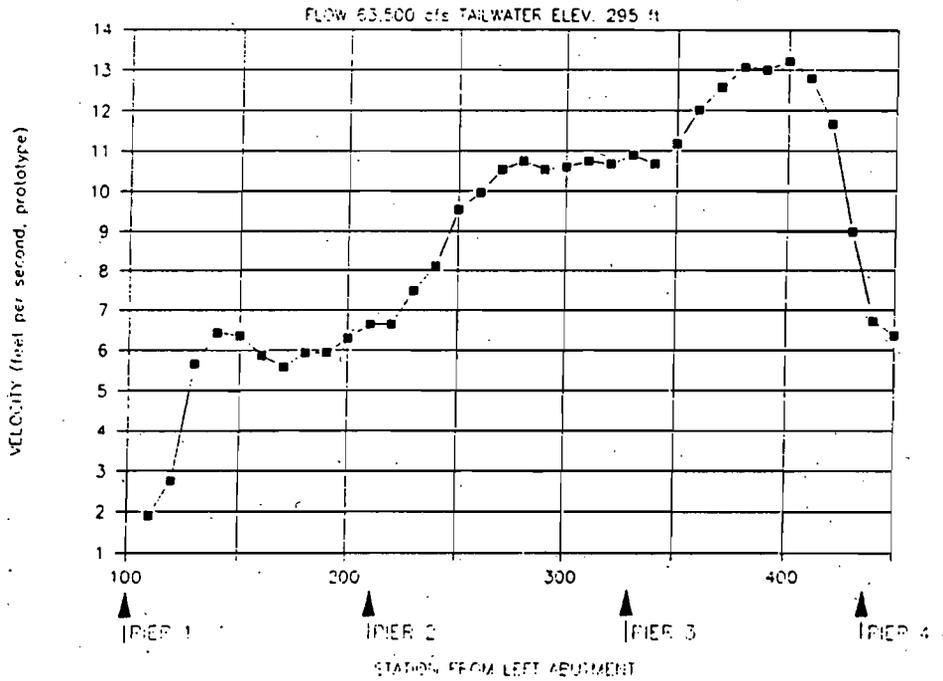


FIGURE D.7. Velocity profile for 63,500 cfs and a 295 feet tailwater elevation.

SCHOHARIE CREEK VELOCITY CROSS SECTION

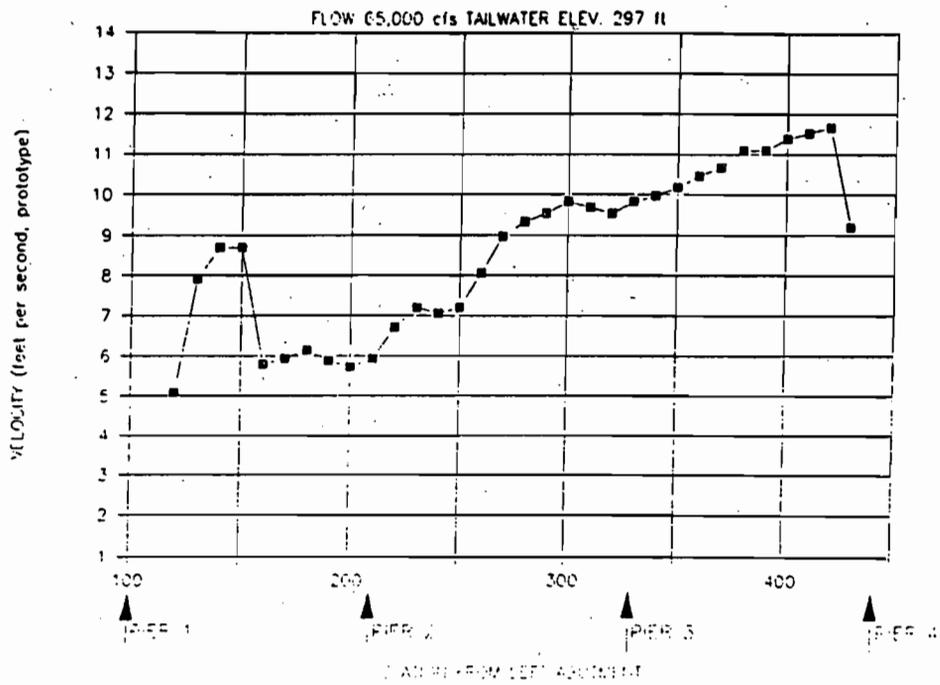


FIGURE D.8. Velocity profile for 65,000 cfs and a 297 feet elevation tailwater.

SCHOHARIE CREEK VELOCITY CROSS SECTION

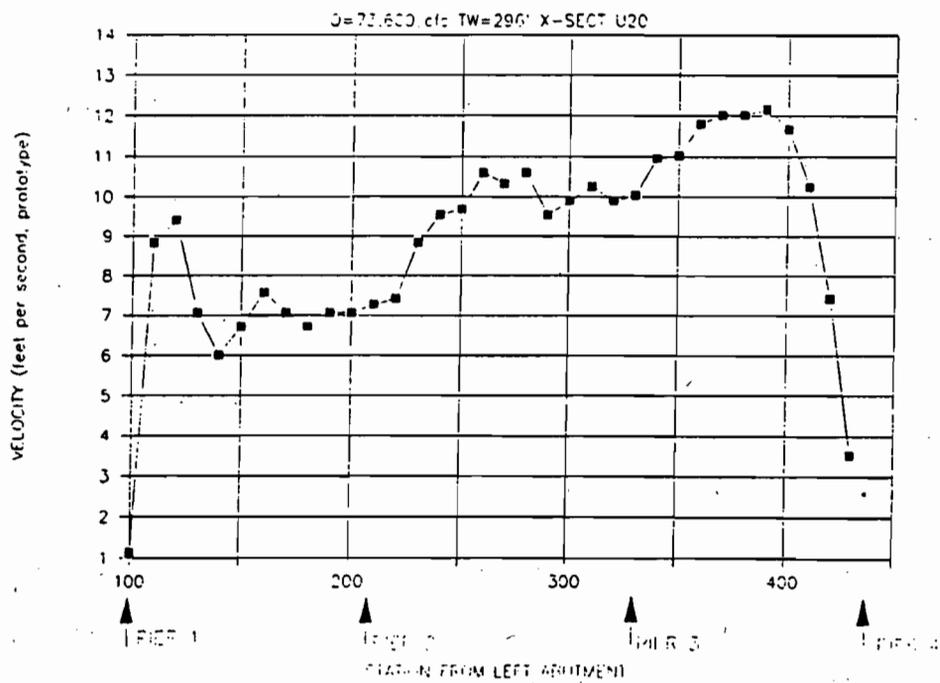


FIGURE D.9. Velocity profile for 73,600 cfs and a 296 feet elevation tailwater, the 1955 flood.

SCHOHARIE CREEK VELOCITY CROSS SECTION

Q=30,000 cfs TW=290' X-SECT U20

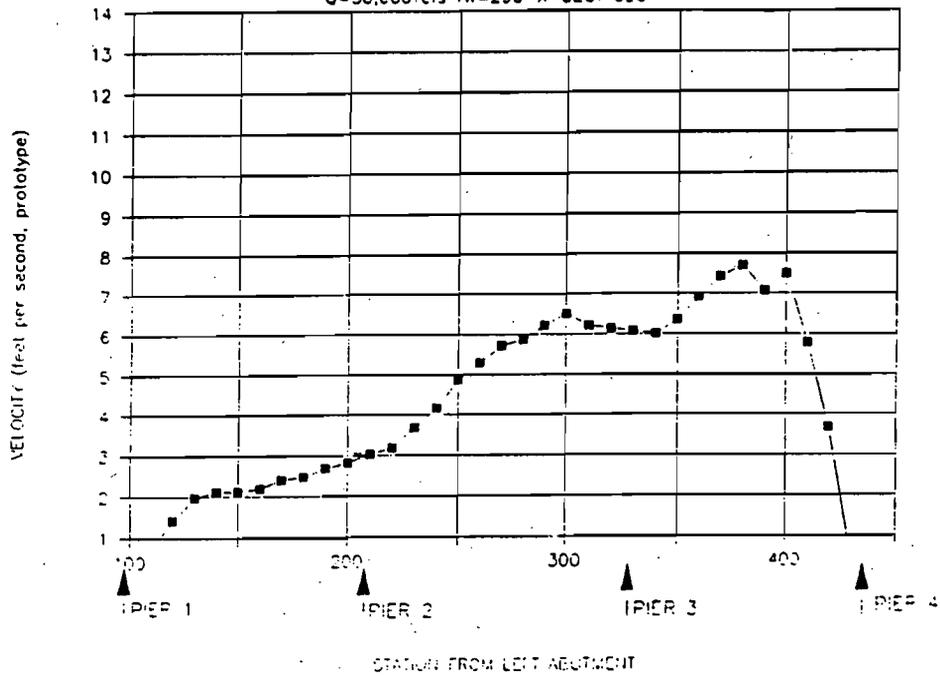
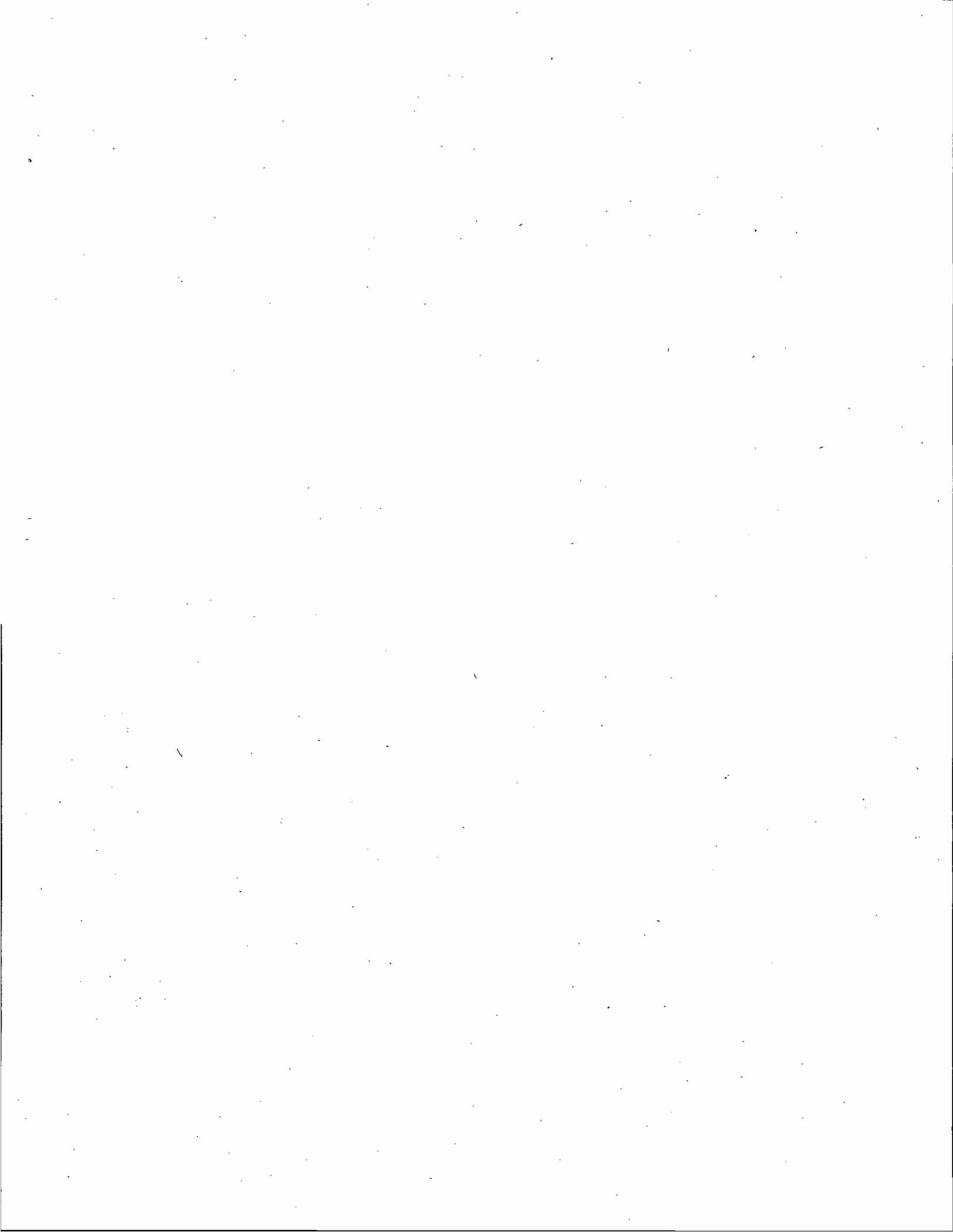


FIGURE D.10. Velocity profile for 30,000 cfs and a 290 feet elevation tailwater, the bank full condition.



## APPENDIX G

NTSB RESPONSE TO NOTICE OF PROPOSED RULEMAKING  
(FHWA DOCKET NO. 87-10)

Office of the Chairman

**National Transportation Safety Board**

Washington, D.C. 20594

JUL 8 1987

Federal Highway Administration  
Room 4205, HCC-10  
400 Seventh Street, S. W.  
Washington, D. C. 20590

Attention: Docket No. 87-10

Dear Sir:

The National Transportation Safety Board has reviewed your Notice of Proposed Rulemaking (NPRM), "National Bridge Inspection Standards; Frequency of Inspection and Inventory," Docket No. 87-10, which was published at 52 FR 11092 on April 7, 1987.

Since the Federal Highway Administration's (FHWA) original issuance of an NPRM on the subject of bridge inspection on January 20, 1983, the Safety Board has investigated two accidents involving collapses of highway bridges. These accidents were on I-95, at Greenwich, Connecticut (Mianus River), on June 28, 1983, and on U. S. 43 near Mobile, Alabama (Chickasawbogue Creek), April 24, 1985. Additionally, we are currently investigating the collapse of the bridge on I-90 near Amsterdam, New York (Schoharie Creek), which occurred on April 5, 1987. In the collapse of the bridges over the Mianus River and Chickasawbogue Creek, the quality of the inspections were of more concern to the Safety Board than the frequency of the inspections since the bridges had been inspected at intervals of nine months and 21 days, respectively, prior to their collapse. Issues involving bridge inspection are also being reviewed by the Safety Board in the collapse of the Schoharie Creek Bridge.

The Safety Board believes that the NPRM is inappropriate in view of the unresolved issues related to the collapse of the Schoharie Creek Bridge and other bridge failures that have recently occurred in the northeastern United States. In fact, there may be specific bridges with unique designs which require inspections at intervals of less than two years to assure the public safety. This NPRM does not even address this possibility.

-2-

In addition, the Board suggests that FHWA develop a set of specific criteria for establishing the proper inspection interval. We believe the five general categories of factors identified by FHWA for potential consideration in determining the prudence of longer intervals ("past experience, age, condition, type/frequency of traffic volume, other relevant factors") are not sufficiently described in the NPRM to elicit comments in areas of Safety Board concern such as inspection of bridges after floods, earthquakes, damage from collisions with marine vessels, changes in either stream configuration or environment, or changes in water quality which would provide the FHWA, with sufficient information to determine the proper inspection interval. The Safety Board also believes that the bridge footing/stream bed interface phenomenon (scour) and the depth and thoroughness of inspection should be criteria for a decision on establishing the proper inspection time interval.

The aforementioned NPRM proposed criteria also do not differentiate between the various types of bridges and the environment to which the bridges are exposed. Additionally, the criteria make no distinction between bridges over land, over water, or over other structures.

Therefore, the Safety Board suggests that the information gathered from the NPRM be utilized to establish proper inspection intervals on all bridges and not as a vehicle to arbitrarily extend the existing inspection time interval.

The Safety Board appreciates the opportunity to comment on this proposal.

Respectfully yours,

ORIGINAL SIGNED BY  
JIM BURNETT

Jim Burnett  
Chairman

## APPENDIX H

DOT INSPECTOR GENERAL RECOMMENDATIONS TO  
FEDERAL HIGHWAY ADMINISTRATION (FHWA)  
AND FHWA RESPONSES

The DOT Inspector General made the following recommendations regarding FHWA deficiencies in its oversight of the States' compliance with the NBIS:

1. Revise the standards to include requirements for underwater inspections.
2. Seek changes to the standards or AASHTO and FHWA bridge inspection manuals to assure that the states (1) establish and document policies, procedures and internal controls for conducting bridge inspections; (2) properly supervise bridge inspectors; (3) conduct periodic management reviews to promote compliance with established state bridge inspection procedures; and (4) prepare complete, accurate and uniform inspection reports. At a minimum, inspection reports should (1) document time spent, equipment needed and used; (2) identify fracture critical bridge members; (3) document changes in ratings; and (4) provide an accurate description of deficiencies and recommendations.
3. Emphasize to the states the need to conduct thorough and well-organized bridge inspections which includes the use of all necessary equipment to assure that all major bridge elements are covered.
4. Require the states to document bridge inspection files with data concerning all corrective actions taken on deficiencies reported by bridge inspectors.
5. Monitor corrective actions taken in FHWA's Region 1 by New York for accomplishing formal load rating analysis and assure that South Carolina complies with the load rating requirements of the standards.
6. Review FHPM 6-4-3-1 or other appropriate guidance to include more specific guidelines on what division offices should cover when reviewing a state's bridge inspection program. As a minimum, together with the nine elements already suggested in the "Maintenance Review Manual," the guidelines should provide for reviews of underwater inspections, internal controls, and corrective action taken on inspector's findings.
7. Require the division offices to prepare written reports and obtain written responses from states on deficiencies identified by FHWA's annual reviews. The responses should indicate the corrective action planned or taken.

8. Establish procedures to assure that the regions' and divisions' monitoring of states' bridge inspection programs are periodically evaluated. Procedures should also be developed to assure that the bridge inspection section of the Annual Maintenance Reports contain specific information to the major elements of the states' bridge inspection program including details of any process reviews performed.

FHWA management response to the Inspector General's recommendations were as follows:

1. FHWA is in the process of evaluating comments received on the notice of proposed rulemaking which proposed changes in the standards to include requirements for underwater inspections. FHWA expects the revised standards to be published in the Federal Register by January 1988. FHWA noted that at least two-thirds of the states have underwater inspection programs well underway, and the remaining states are actively establishing their programs.
2. An FHWA program official indicated that, although not specifically included in the Headquarters policy memorandum dated October 26, 1987, to the regional offices, emphasis in division office reviews will be placed on all four items included in our recommendation. These items, except for states conducting periodic management reviews of bridge inspection procedures and documenting the time spent inspecting bridges, are presently included in either the standards, the notice of proposed rulemaking, the AASHTO "Manual for Bridge Maintenance Inspection" or FHWA's bridge inspection manual. However, the program official stated that FHWA plans to write to AASHTO in FY 1988 suggesting that states conduct periodic management reviews to promote compliance with established state bridge inspection procedures. While FHWA indicated that documenting the time spent on inspections may be valuable to managers of a bridge inspection team, they did not agree that inspectors must be required to document time spent inspecting each bridge.
3. FHWA has continually emphasized to the states, since the program began, the need to conduct thorough and well organized inspections. A headquarters policy memorandum issued to all Regional Federal Highway Administrations on October 26, 1987, identified the areas in the bridge program to be emphasized in FY 1988. According to the memorandum, division offices were directed to provide emphasis on the thoroughness of inspections and inspection equipment in their reviews of states' bridge inspection programs.

4. FHWA indicated that the current standards and the notice of proposed rulemaking which would change the standards require that states document corrective action taken as a result of bridge inspection recommendations. Also FHWA headquarters policy memorandum issued on October 26, 1987, provides that each FHWA region will have a program to ensure that FHWA division offices place emphasis on evaluating the documentation of followup action on bridge inspection reports prepared by the states.
5. The FHWA has been working on a day-to-day basis with the States of New York and South Carolina to ensure that they load rate all of their bridges.
6. FHPM 6-4-3-1, which includes requirements for division offices to conduct annual review of state bridge inspection programs, is being revised to include major program elements to be reviewed. FHWA plans to issue the revised FHPM in January 1988. In addition, according to FHWA headquarters policy memorandum issued to the regions on October 26, 1987, each division office will be expected to have comprehensive review guidelines to conduct bridge program reviews. FHWA indicated that such guidelines should cover the nine elements suggested in the FHWA "Maintenance Review Manual" as well as specialized inspections including inspections of fracture critical bridges and underwater inspections. The FY 1988 program guidance provides that each division office also place emphasis on evaluating state followup action taken on inspectors' findings. In discussions held with an FHWA headquarters program official, subsequent to the issuance of the FHWA policy memorandum dated October 26, 1987, we were advised that the program reviews made by FHWA Divisions would also include evaluations of the states' internal controls over bridge inspections.
7. FHWA's headquarters policy memorandum dated October 26, 1987, provides that each region should have a procedure to ensure that (1) all findings stemming from standards reviews result in written regional and division office reports, (2) states be informed of FHWA's findings in writing, and (3) findings are resolved.
8. According to the October 26, 1987, policy memorandum, FHWA headquarters plans to participate in at least one bridge management review in each region in FY 1988. Headquarters emphasis during these reviews will focus on how effective the regional programs are in evaluating and bringing about improvements in the division office reviews of the states' bridge inspection program. With respect to the second part

of the audit recommendation, FHWA indicated that the annual evaluation report on state bridge inspection programs will be part of the impending revision to appropriate FHPMs [Federal Highway Program Manuals]. The revised FHPMs will require both an evaluation of the major elements of state bridge inspection programs and corrective actions taken to overcome weaknesses. These evaluations provide support for the information included in the annual reports.