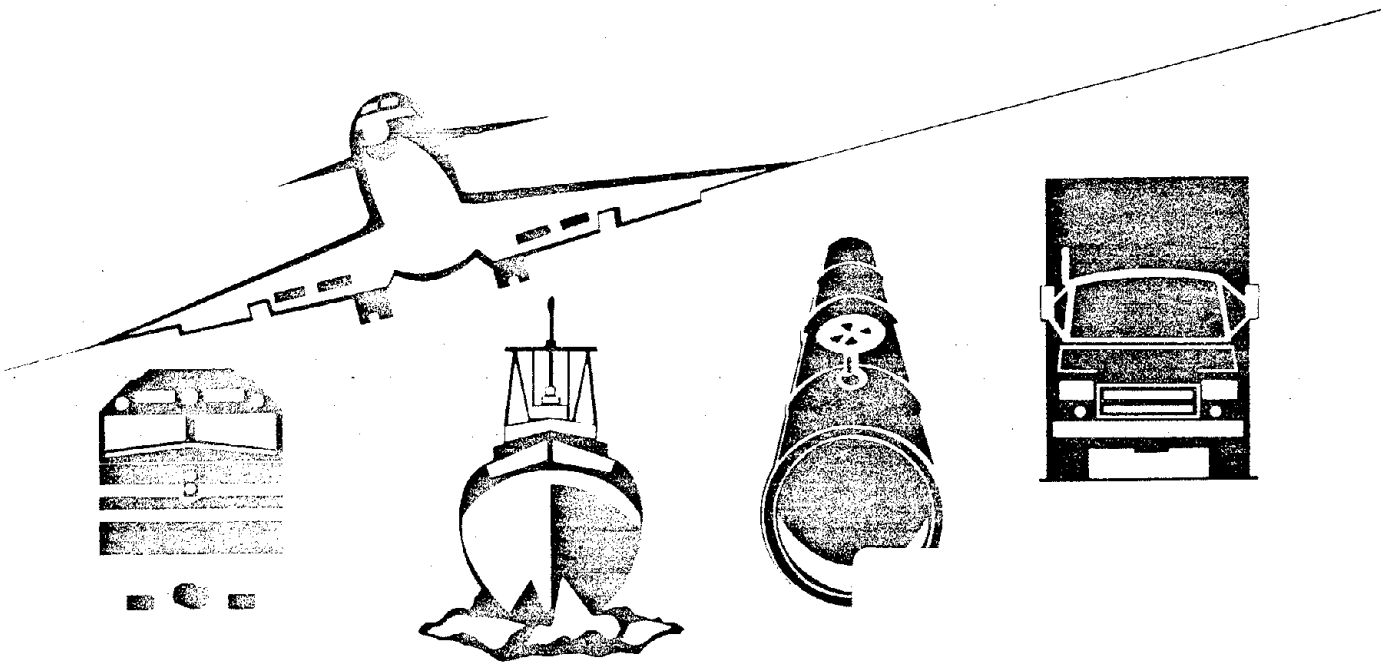
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NTSB/HAR-90/03

NATIONAL TRANSPORTATION SAFETY BOARD

HIGHWAY ACCIDENT REPORT

COLLAPSE OF THE
HARRISON ROAD BRIDGE SPANS
MIAMITOWN, OHIO
MAY 26, 1989



REPRODUCED BY:
U.S. Department of Commerce
National Technical Information Service
Springfield, Virginia 22161

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16. Abstract

This report explains the collapse of a section of a temporary bridge over the Great Miami River in Miamitown, Ohio, on May 26, 1989. The safety issues discussed in the report are temporary and permanent bridge design and procedures for closing temporary and permanent bridges. Safety recommendations addressing these issues were made to the Hamilton County (Ohio) Engineer's Office, the Ohio Department of Transportation, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the United States Geological Survey.

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EXECUTIVE SUMMARY

On May 26, 1989, about 5:25 p.m. eastern daylight time, a 140-foot section of the 556-foot Harrison Road temporary bridge over the Great Miami River fell about 40 feet into the rain-swollen river after a pile bent collapsed. Seven witnesses reported that a passenger car and a pickup truck fell into the river. However, only a passenger car and the bodies of the car's two occupants have been recovered from the river. No other vehicles were found in the river nor are any persons reported missing in the Miamitown area. Witnesses reported an unusual amount of debris floating down the river and striking the pile bents of the bridge prior to the collapse. Although the weather was clear and dry, flooding conditions existed at the time of the collapse and the river had overflowed its banks onto the flood plain.

The National Transportation Safety Board determines that the probable cause of the temporary bridge collapse was the selection by the Hamilton County Engineer's Office of a design by the National Engineering and Contracting Company that did not consider lateral loads and the failure of the Hamilton County Engineer's Office to promptly close the bridge when it became subject to significant debris loading. Contributing to the cause of the collapse was the failure of the Hamilton County Engineer's Office to submit the bridge design plans to the Ohio Department of Transportation for review as required by State law.

The primary safety issues raised by this accident are the adequacy of the temporary bridge design, the adequacy of procedures for closing temporary and permanent bridges under conditions that exceed bridge design criteria, and the adequacy of current design guidelines and specifications for highway bridges, particularly for debris loading.

Safety recommendations addressing these issues were made to the Hamilton County Engineer's Office, the Ohio Department of Transportation, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the U.S. Geological Survey.

NATIONAL TRANSPORTATION SAFETY BOARD
WASHINGTON, D.C. 20594

HIGHWAY ACCIDENT REPORT

COLLAPSE OF THE HARRISON ROAD BRIDGE SPANS
IN MIAMITOWN, OHIO
MAY 26, 1989

INVESTIGATION

Events Prior to the Collapse

About 4:20 p.m. on May 26, 1989, a project superintendent employed by National Engineering and Contracting Company (NECC), the designer and contractor of the one-lane temporary bridge over the Great Miami River at Miamitown, Ohio (the Harrison Road bridge), stopped at the bridge[1] site on his way home from another work project. He had heard that construction workers at the bridge site had moved a large crane away from the river because of high water, and he wanted to make certain that it had been moved to a safe location. While there, he checked the river gauge, noticed debris coming down the river, walked out on the temporary bridge, and looked down between the pickup holes in the mats[2] to see the accumulation of debris at the pile bents.[3] He stated that he saw two refrigerators and large trees floating down the river. He also saw a telephone pole with a tree leaning against it on the east bank north (upstream) of the bridge.

The superintendent sighted the upstream bridge curb line and noted no deflection. He walked to the west bank, went underneath the bridge, looked around, and observed debris wedged against the pile bents in the river. He noticed a tree about 18 inches in diameter on the east side of pile bent 2 (parallel to it) with the roots upstream and the limbs downstream and saw a floating pole strike the roots of the tree. While underneath the bridge, he felt the steel beams but did not detect any vibration. The superintendent then walked back onto the bridge and noticed a telephone pole that was embedded in the ground but leaning toward the bridge with a phone cable suspended over the traveled portion of the bridge. (See figure 1.) He again read the river gauge. He stated that although he had heard a news broadcast that the river was supposed to crest at 20 feet, the gauge indicated almost 21 feet.

About 4:48 p.m., the superintendent went into the NECC field office, a construction trailer on the east bank, and notified his supervisor by telephone about the river level and the leaning telephone pole. He also called the hydrologist at the River Forecast Center (RFC) in Cincinnati, Ohio, and was advised that the river crest was being revised to 23 feet. He again called his supervisor and told him that he was going to call the Hamilton County Engineer's (HCE's) Office; he was concerned that

---

[1] A temporary bridge had been constructed before the old truss bridge was closed and removed; a new permanent bridge was being constructed.

[2] Large hardwood timbers (10" x 10" x 24') joined together in 4-foot sections that made up the bridge deck.

[3] A pile bent is a pier composed of driven piles. This particular pile bent consisted of four steel piles and was capped at the top with a steel beam to form a bridge seat.



Figure 1.--Temporary bridge (precollapse condition) looking west to east.

the telephone pole and line would fall on the traffic crossing the bridge and believed that the bridge should be closed as a precautionary measure. In addition to notifying the telephone company, he directed an NECC carpenter foreman to report to the site to build a brace to keep the telephone pole from falling onto the bridge. The superintendent then called the HCE bridge engineer to obtain approval to close the bridge.

The HCE bridge engineer, who had just returned from vacation, was lying down because he was suffering from a migraine headache. He said that he took the call between 5:00 and 5:15 p.m. The bridge engineer stated that he told the superintendent, "I will take the authority for you to close the bridge down." He added, "I said to him, I will call [the HCE], tell him that I have authorized the closure of this bridge, just to notify him, because I knew the press would be calling [the HCE]." The bridge engineer was unable to reach the HCE but was able to contact the Chief Deputy HCE. The deputy received the bridge engineer's telephone call concerning closure of the bridge at 5:17 p.m.

The superintendent stated that when he called the bridge engineer and explained the problem, the bridge engineer said:

> . . . he guessed he could give me the authority or I could call the police or he said you would have to talk to [an HCE employee] about getting the signs to close the bridge. Then he said let me call [the HCE] and then I'll call you back.

About 5:25 p.m., the bridge engineer returned the superintendent's call; while the superintendent was talking on the telephone in the construction trailer, he heard a loud bang, looked out a trailer window, and saw part of the west deck of the bridge go straight down. He called "911" and reported the bridge collapse.

A witness on the west bank stated that about 5:15 or 5:20 p.m. he saw a huge amount of debris, including a boat, part of a dock, and many large trees with roots attached, coming down the river. He said the pile of debris was about 10 feet high and that he saw it strike pile bent 2. (See figure 2.) About the same time, another witness at the same location saw the same debris and estimated that the trees that had accumulated on pile bent 2 were "back upriver about 30 to 40 feet." Both witnesses saw the debris strike pile bent 2, heard a loud noise, and saw two spans fall into the river.

The Accident

Two spans and pile bent 2 fell into the rain-swollen Great Miami River, which had risen above its flood stage and had extended onto the adjacent flood plain. Traffic was traveling westbound on the one-lane bridge at the time of the collapse. A blue 1977 Mercury Cougar, with a driver and one passenger, fell into the river. The driver of the passenger car following the Mercury observed the car and bridge spans fall together into the river. She stated that she immediately put her car in reverse and backed up as far as she could while motioning for the drivers of vehicles behind her to back up. She stated that as she backed up, the wooden mats making up the bridge deck were dropping in sections in front of her. Another motorist stated that she was almost three-quarters of the way across the bridge when she heard a noise and her car started swaying back and forth. She thought that her tires were going flat, so she accelerated to get off the bridge. As she reached the west end of the bridge, she looked in her rearview mirror and saw the Mercury fall in the river.

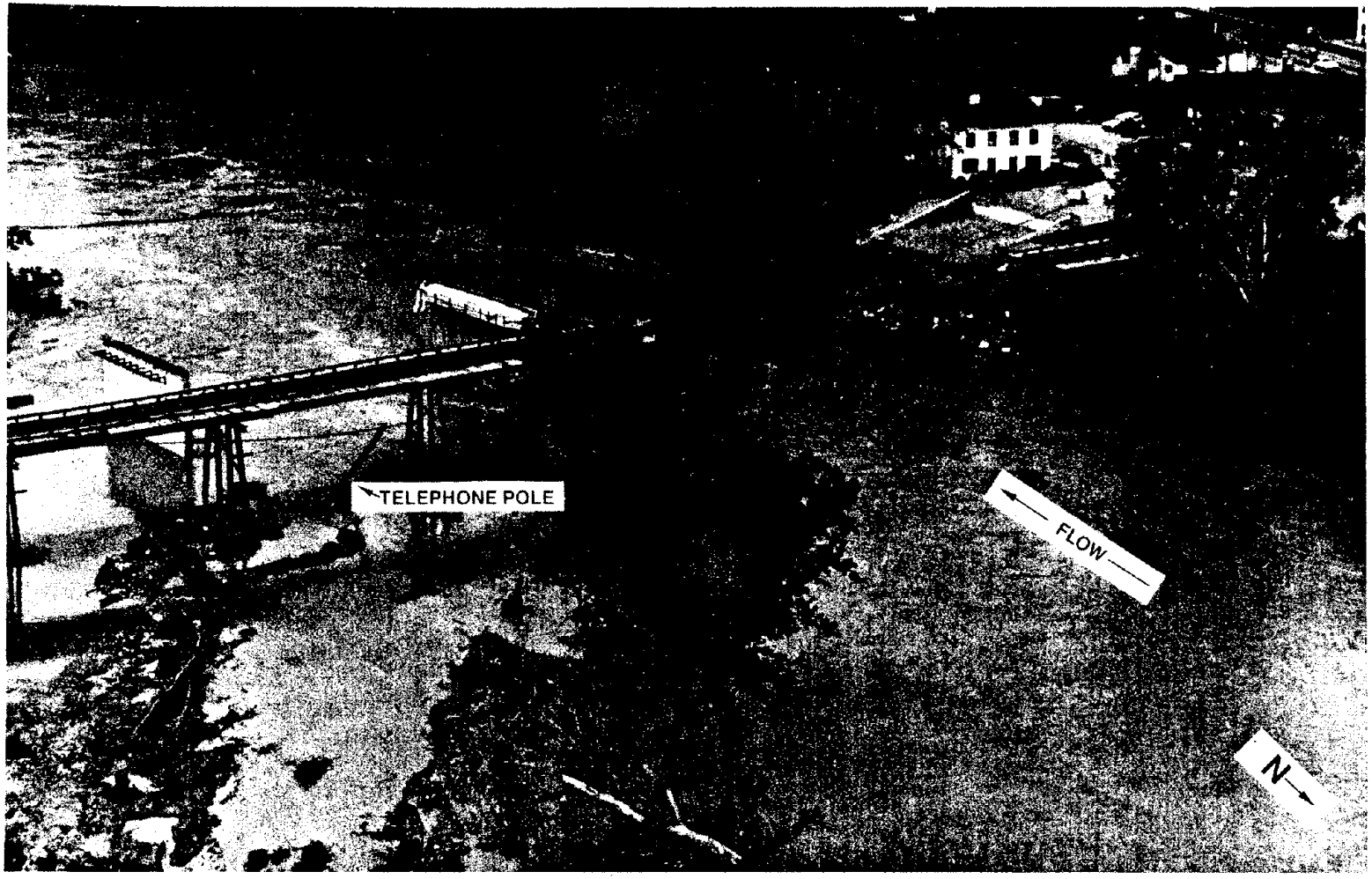


Figure 2.--Sections of the temporary bridge (precollapse condition).

Seven witnesses (five on the bridge and two on the west bank) reported that a passenger car and a pickup truck fell into the river. Only the passenger car (the Mercury) was recovered from the river, and no other persons have been reported missing in the Miamitown area. Several witnesses commented about the height and turbulence of the river in their statements to the police.

Emergency Response and Search

The Hamilton County Communications Center received several "911" calls, and emergency response units were dispatched at 5:27 p.m. from the Hamilton County Sheriff's Office (HCSO), the Whitewater Township Fire Department, and the Colerain Township Fire Department. Following the collapse, many witnesses searched the river for signs of the car, the pickup truck, and their occupants, but no one saw the vehicles after they entered the water. By 6:30 p.m., an Ohio Highway Patrol helicopter, as well as mounted and foot patrol emergency response personnel, were searching for the victims along both banks downriver of the bridge. The river level and the water velocity prevented boat patrols and divers from searching until 2 days later. The search and recovery efforts lasted 9 days and included 60 to 80 volunteers, 4 fire engines, 3 emergency medical service units, 6 boats, 6 divers, 1 helicopter, and sonar equipment.

Vehicle and Occupant Information

One body (passenger car driver) was recovered 4 days after the collapse approximately 5 miles downstream. Eight days after the collapse, with the aid of a sonar device, the submerged passenger car was located about 300 feet downstream of the temporary bridge and 15 feet from the west bank. The roof of the car was substantially crushed and both car doors were jammed shut. (See figure 3.) The passenger's body was found in the rear passenger compartment of the car. The medical examiner found no blunt trauma injuries and ruled that the cause of both deaths was "asphyxia due to drowning."

Injuries[4]

	Drivers	Passengers	Total
Fatal	1	1	2
Serious	0	0	0
Minor	0	0	0
None	0	0	0
Total	1	1	2

Highway Information

Harrison Road is a two-lane asphalt roadway that runs east-west parallel to Interstate 74 (I-74). It is the main route between Whitewater Township and Colerain Township serving Miamitown, Ohio. The road had been designated U.S. Route 52 until 1974, when the State turned over the road and the existing bridge to Hamilton County and the roadway was redesignated County Route 457. The posted speed limit for the road was 45 mph; however, near the bridge, construction signs were

[4]See appendix B for the Abbreviated Injury Scale.

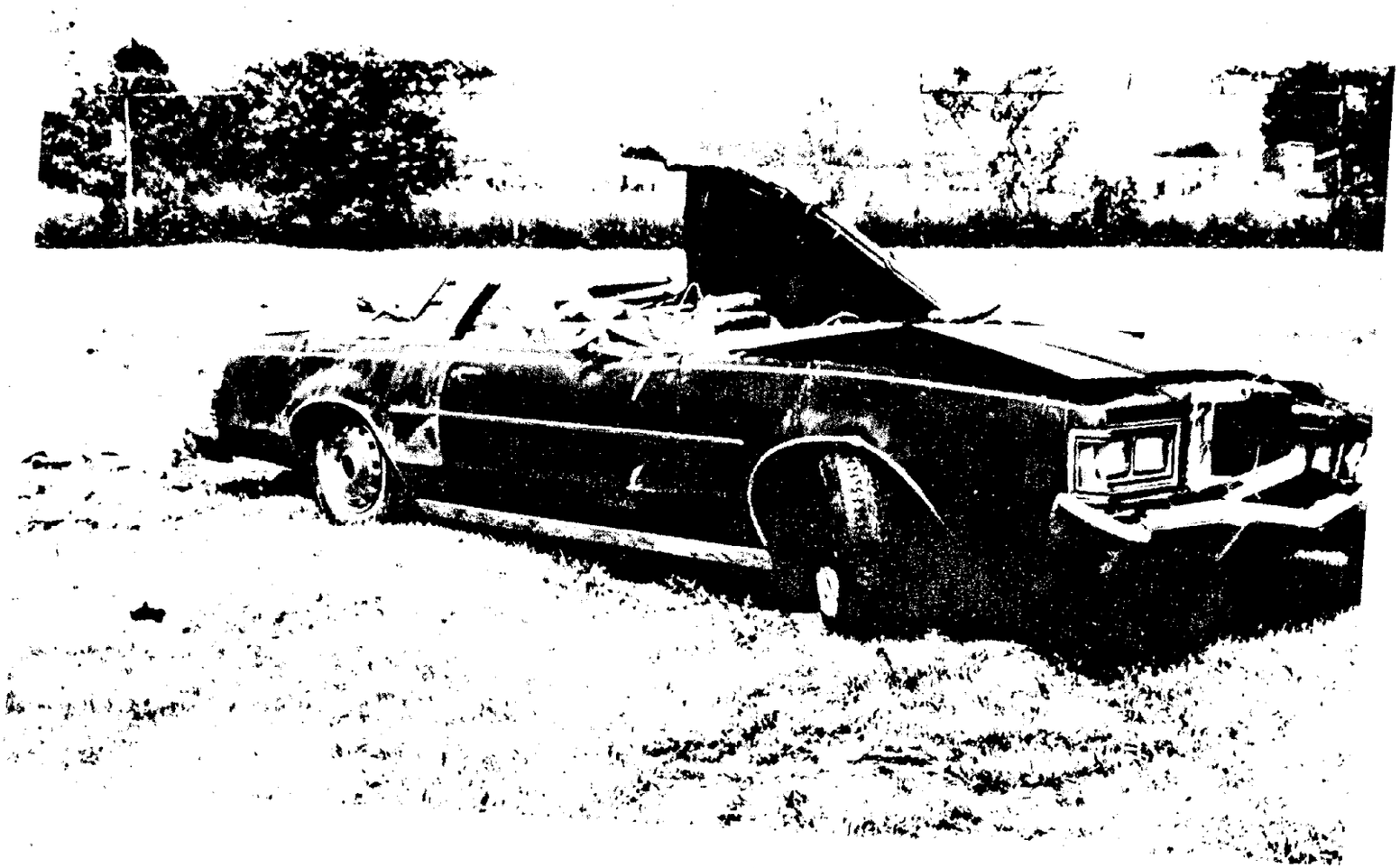


Figure 3.--1977 Mercury Cougar.

posted and the speed limit had been reduced to 25 mph. Advisory signs indicated: "ONE-LANE BRIDGE," "TRAFFIC SIGNAL," and "NO TRUCKS, 4-WHEEL VEHICLES ONLY, EXCEPT EMERGENCY VEHICLES." The approach roadways were delineated with solid white edgelines and a solid double yellow centerline, and the roadway curved to the left for westbound traffic. The average daily traffic count for Harrison Road was about 3,000 vehicles. (See figure 4.)

Traffic signals were installed at both ends of the temporary bridge to manage the alternate one-way traffic flow. The signals changed on a fixed time cycle, with 20-second green indications for each direction of traffic and red indications for opposing traffic. Each green indication was followed by a 3-second yellow and a 22-second all-red interval, which permitted the bridge traffic to clear.

The traffic control box and the police emergency box were located on the west end of the bridge. Neither the HCE, nor the county bridge engineering staff, nor NECC personnel had keys to either box. Personnel from the county's traffic engineering section had keys to both boxes. The sheriff's department had keys to the police emergency box.

Bridge Information

History of Bridge Site.--In 1894, the King Bridge Company built a 440-foot, single-span, through-truss bridge[5] over the Great Miami River at Miamitown, Ohio. In 1974, major repairs and modifications were made to the bridge, and later that year, the State of Ohio turned the highway and the bridge over to Hamilton County. According to the Miamitown Businessman's Association (MBA), when the truss bridge was closed for repairs for 16 months in 1974 and 1975, many businesses faltered and some went bankrupt.

In 1987, because of the deteriorated condition of the truss bridge, the HCE decided to replace it with a new bridge at the same location. On May 6, 1987, the HCE hired a consultant, Graham, Obermeyer & Partners, Ltd. (GOP), to design a replacement bridge. Harrison Road was scheduled to be closed for approximately 1 year during construction of the replacement bridge, and a proposed detour for local traffic involved traveling about 2 miles farther west on I-74 and exiting at Route 128, then traveling about 1/2 mile north to Miamitown. This detour takes about 10 minutes. Because of the financial hardship anticipated owing to the closure of the Harrison Road bridge, the MBA petitioned the HCE to build a temporary bridge. The HCE agreed to do so to accommodate local traffic until the permanent bridge was open to traffic.

GOP's Temporary Bridge Design Plans.--On April 6, 1988, the HCE signed an agreement with GOP to design a temporary bridge for the site.[6] The plans prepared by GOP indicated multiple simple spans (concrete box beam girders) supported by five tower-frame bents and two abutments. Each tower bent was to consist of ten HP 12 x 74 steel sections arranged in two rows of five each. The six inside HP sections were to be battered longitudinally, and the four corner piles were battered both

---

[5]A jointed structure with an open web construction so arranged above the roadway that the frame is divided into a series of triangular figures with its component straight members stressed axially only.

[6]According to the HCE, consultants hired by the HCE's office perform 90 percent or more of the work on bridges and large roadway projects

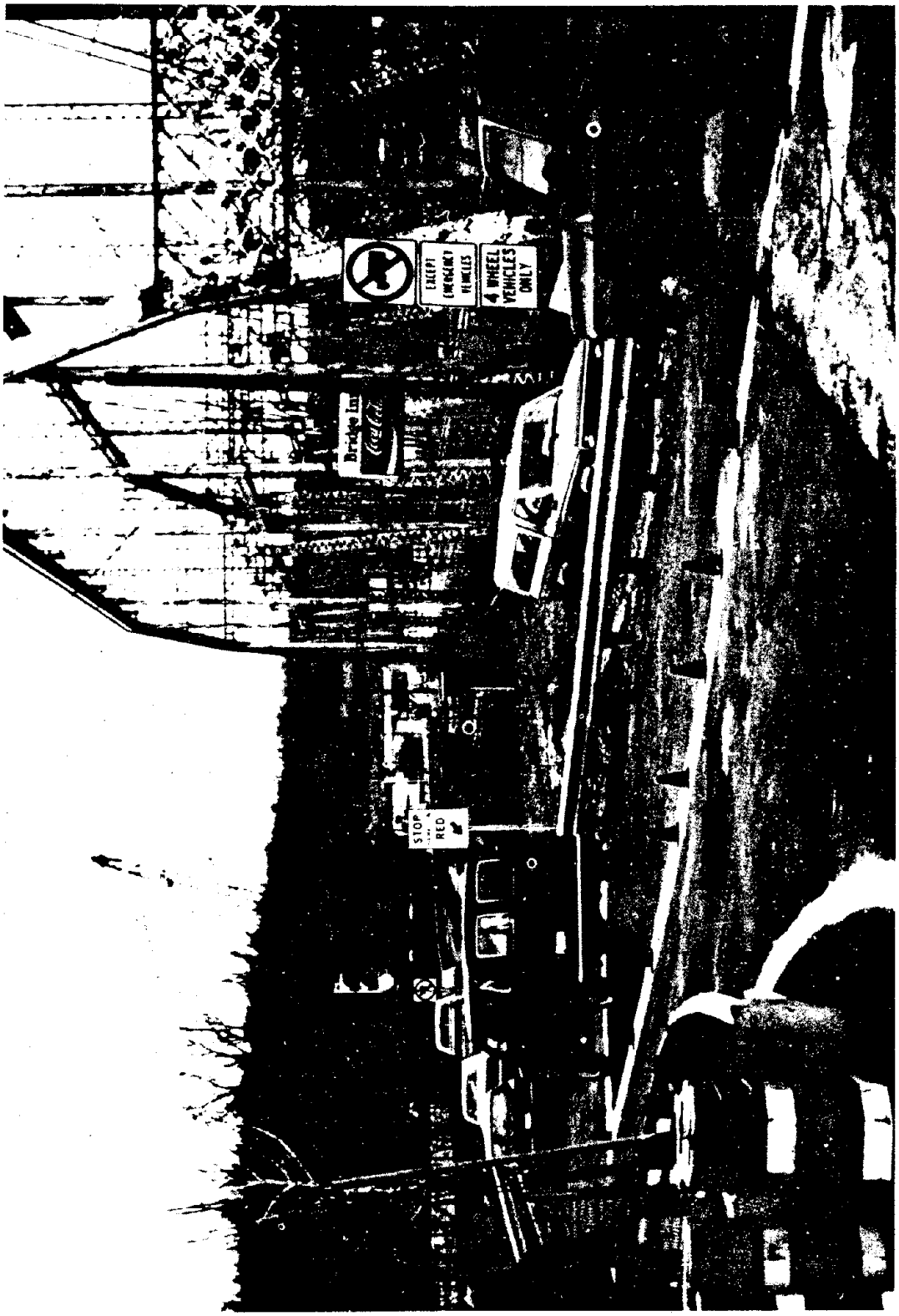


Figure 4.--Old truss bridge and temporary bridge.

longitudinally and laterally. (See figure 5.) The piles were to be connected using diagonal and horizontal bracing with bolted connections. The bottom horizontal brace was to be placed as close as possible to the water level for pile bents 1 and 2. From 2 feet above the bottom horizontal brace to 2 feet above the riverbed, cable bracing positioned in an "X" configuration was to be attached in both the lateral and longitudinal directions. The piles were to be driven to refusal on bedrock and embedded at the top in a reinforced concrete pile cap. The box beam spans would later be transferred from the temporary bridge to the replacement bridge for use as spans in the replacement bridge. Other than a partial cross section of the riverbed, the GOP site plans for the permanent and temporary bridges depicted no hydrological or hydraulic information. The GOP design calculations did determine lateral loads due to stream flow using a stream velocity of 10 feet per second (fps).

Selection Process for the Replacement and Temporary Bridges.--In August 1988, seven contractors submitted sealed bids to erect a temporary bridge based on the GOP design, dismantle the truss bridge, and construct a replacement bridge over the Great Miami River. The HCE estimate for the bridge project was $2,438,284.56. NECC submitted the lowest bid, $2,297,141.20, and was awarded the contract, which specified that the contractor "shall start work on the temporary bridge within 10 days after signing the contract and that the temporary bridge shall be open to traffic within 200 days of signing the contract." The contractor had 500 calendar days from the inception of the contract to completion, including completion of the replacement bridge and removal of the temporary bridge. The contract between the HCE's office and NECC had no provisions for monitoring or closing the temporary bridge.

In September 1988, NECC submitted an alternate design for the temporary bridge different from the one developed by GOP. The alternate design and construction project had an anticipated cost savings to the HCE's office of about $48,000 and a time savings of about 6 months for completion of the permanent bridge. In a letter dated October 5, 1988, NECC offered to accept liability for design of the temporary bridge if the HCE approved use of the NECC alternate temporary bridge design. The HCE retained GOP to review these alternate design plans. A pre-construction meeting on October 6, 1988, included representatives of the HCE, NECC, and GOP; the designer of the NECC temporary bridge was not present. The GOP engineer responsible for reviewing the NECC plans testified at the Safety Board's public hearing that he expressed concern at the pre-construction meeting about the lack of lateral load considerations in the design. He stated that at this meeting he asked about both the lateral design criteria for the bridge and the lateral load transmission for wind and for longitudinal loading because there was no evidence of horizontal cross bracing in a direction parallel with the deck. He said that the NECC representatives told him that this design had been used on at least one other occasion by the contractor and that they had confidence in it but would review his concerns. GOP raised these same concerns in an October 10, 1988 letter to the HCE. The consultant stated:

> (a) National [NECC] has responded that substructure lateral loads are transferred to the abutments and to soil without any formal longitudinal bracing.
>
> (b) Anticipated bearing elevations may need to be adjusted in field, and National seems to understand this.

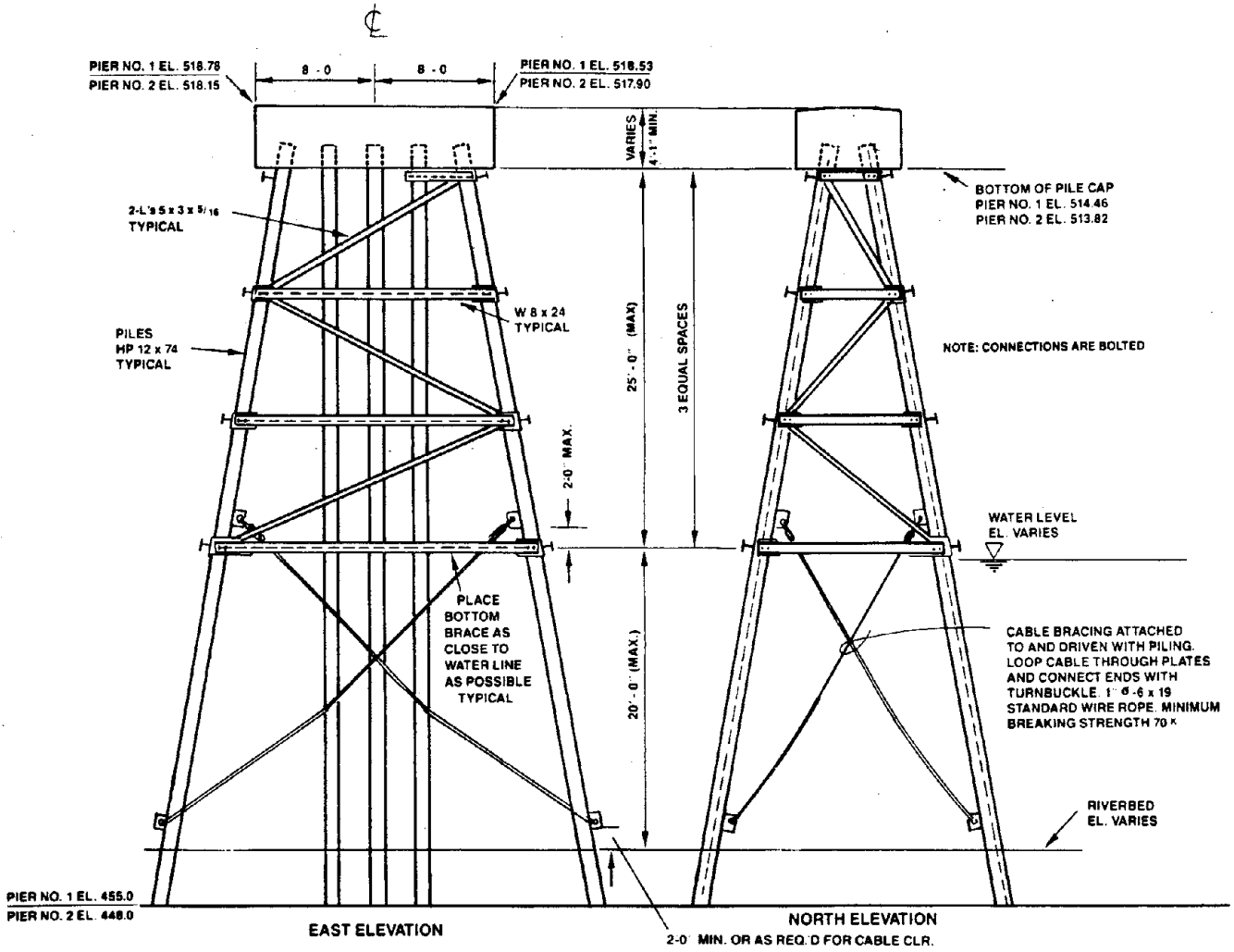


Figure 5.--Schematic of tower bents 1 and 2 as designed by consultant.

(c) Ice, flood, and stream flow (read debris) criteria are not specifically designed into this bridge and National agrees. I guess this is why it's temporary.

(d) Possibly making some of that east end piers closer to the east abutment and potential effects: I've asked H.C. Nutting Co. [geotechnical and testing engineers] to check into this based on forthcoming distance information.

The HCE did not require NECC to make any changes to the design plans for the temporary bridge as a result of the concerns expressed by GOP. On October 19, 1988, the HCE approved the NECC temporary bridge design, and the contractor began work. In December 1988, the temporary bridge was open to traffic.

NECC's Temporary Bridge Design Plans.--The temporary bridge constructed by NECC was designed for dead loads,[7] which consisted of the weight of the H-piles, W-beams, the cross bracing, the timber deck, ice loads on the deck, and the asphalt deck surface. According to the NECC design plans, the temporary bridge was also designed for live loads in accordance with the HS20-44[8] vertical loading. The plans did not indicate that the bridge was designed for lateral loads, nor did they show calculations for lateral loads. The NECC design plans specified cross bracing of the piles to reduce the effective length of the columns, but they included no calculations or details for the cross bracing connections. The plans indicated that the cross bracing was to be placed approximately down to the riverbed; however, the bridge was constructed with the lowest point of the cross bracing 12 feet above that shown on the plans. Also, the upper cross bracing of pile bent 2 did not extend up to the pile cap; instead, it began 3 feet 2 inches below the bottom of the pile cap. (See figures 6 and 7.)

Bridge Design Considerations.--At the Safety Board's public hearing, the NECC designer of the temporary bridge (a registered professional engineer who had 10 years experience with NECC) testified that the temporary bridge was designed in accordance with ASSHTO specifications for HS20-44 vertical loading. In addition to using the AASHTO vertical load criteria, the NECC designer testified that he also referred to the Ohio Department of Transportation (ODOT) Construction and Material Specifications, specifically Item 502. (See section on ODOT Requirements.) When asked by Safety Board investigators whether he had designed the temporary bridge for either impact or lateral (wind, stream flow, or debris) loading, he replied, "I don't believe so." He explained that he had determined that there was a substantial margin of safety for vertical loads (3.5 to 1) in using the 12 x 53[9] H-piles and that he believed this margin of safety allowed for any lateral loading to which the bridge would be subjected. He also stated that he did not consider debris

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[7]A static load from the weight of the structure itself.

[8]The standard American Association of State Highway and Transportation Officials (AASHTO) designation for the live or moving loads to be applied on the deck for determining highway load limits. An HS20-44 vehicle load is a 3-axle, 36-ton tractor-semitrailer with a gross weight of 72,000 pounds.

[9]The first number indicates 12-inch-wide flanges; the second number indicates that the section weighs 53 pounds per linear foot.

loading or the effects of scour[10] in the design of the temporary bridge because he did not consider them major factors.

He also testified that he designed the bridge so that the flanges of the H-piles were parallel to the stream flow and said that he did so to provide a surface for bracing the four piles together. He explained that he was not involved in the construction of the temporary bridge and was not aware of any discrepancies between the design plans and the actual construction. Although his design indicated that the bracing for pile bent 2 was to be attached to the battered piles just above the riverbed, he said that he expected the attachment to be made just above the water level, which he anticipated would be 4 or 5 feet above the riverbed. Because this distance was less than the shortest unbraced length that he used in calculating the safe column loads, he believed the piles would still safely carry the vertical loading for which they were designed.

According to testimony by the NECC designer, soundings of the river to establish riverbed elevations and the mean river flow data were the only hydraulic or hydrologic information used by NECC in the design and construction of the temporary bridge.

Temporary Bridge Construction.--The Harrison Road temporary bridge, which opened for local traffic in December 1988, consisted of nine simple spans[11] that were supported by eight pile bents and two concrete abutments. The abutments on both ends of the bridge consisted of reinforced concrete spread footings supported by soil embankments. The temporary bridge was 68 feet north of the old truss bridge centerline and had a curved section at the east end transitioning toward the existing alignment of Harrison Road. (See figure 8.) The pile bents consisted of four steel H-piles in a plane, two vertical and two battered.[12] The plans called for each pile to be driven to refusal[13] in the riverbed. The H-piles were erected with the web[14] perpendicular to the flow of the river, and the end piles were battered at a slope of six vertical to one horizontal. Two levels of cross bracing had been used to reduce the unbraced length of the piles. The upper level of bracing consisted of two 4-inch x 4-inch x 3/8-inch steel angles with one angle welded to the east flanges of the piles and the other to the west flanges. The lower bracing consisted of two 7-inch x 4-inch x 3/8-inch steel angles with one angle welded to the east flanges and the other to the west flanges. The pile caps consisted of HP 14 x 117 steel sections. (See figure 9.) Each of the nine spans, ranging in length from 54.75 to 70.75 feet, consisted of 36-inch deep steel girders (W-shape), which overlapped slightly over the supports and were welded to the pile caps. The bridge deck was made up of hardwood mats and the wearing surface was covered with asphalt. The mats were

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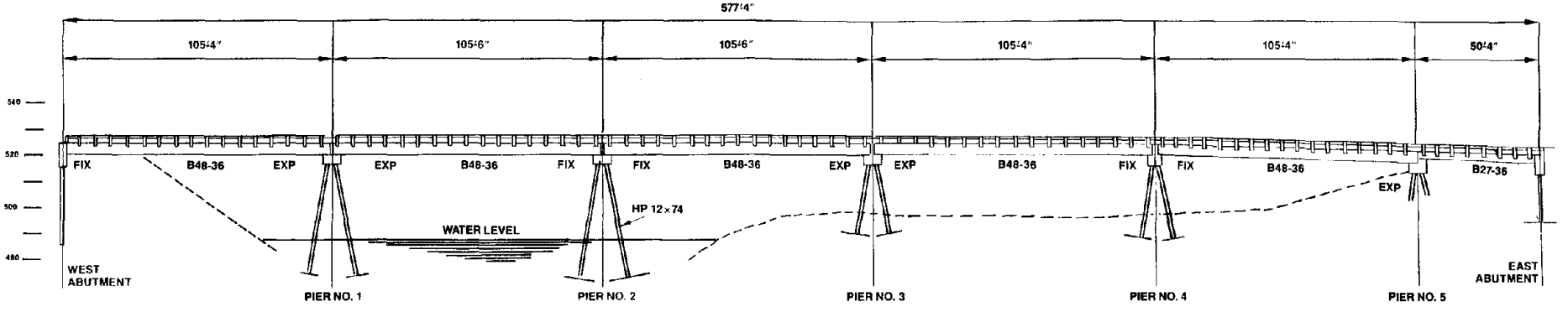
[10]An erosion of a river, stream tidal inlet, lake, or other water bed area by a current, wash, or other water in motion, producing a deepening of the overlying waterway or a widening of the lateral dimension of the flow area.

[11]A superstructure span having, at each end, a single unrestraining bearing or support and designed to be unaffected by stress transmission to or from an adjacent span.

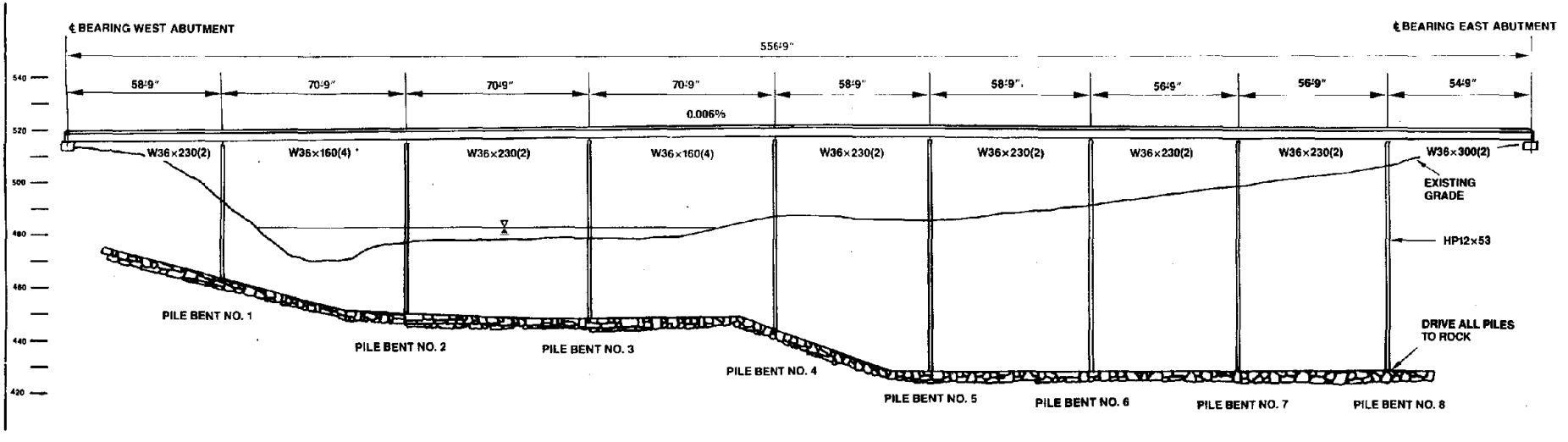
[12]A pile driven in an inclined position to resist forces that act in other than a vertical direction.

[13]Driven to bedrock or until 1 inch or less penetration is achieved with 20 or more blows or impacts of the pile driving hammer.

[14]That part of an H-pile located between and connected to the flanges.



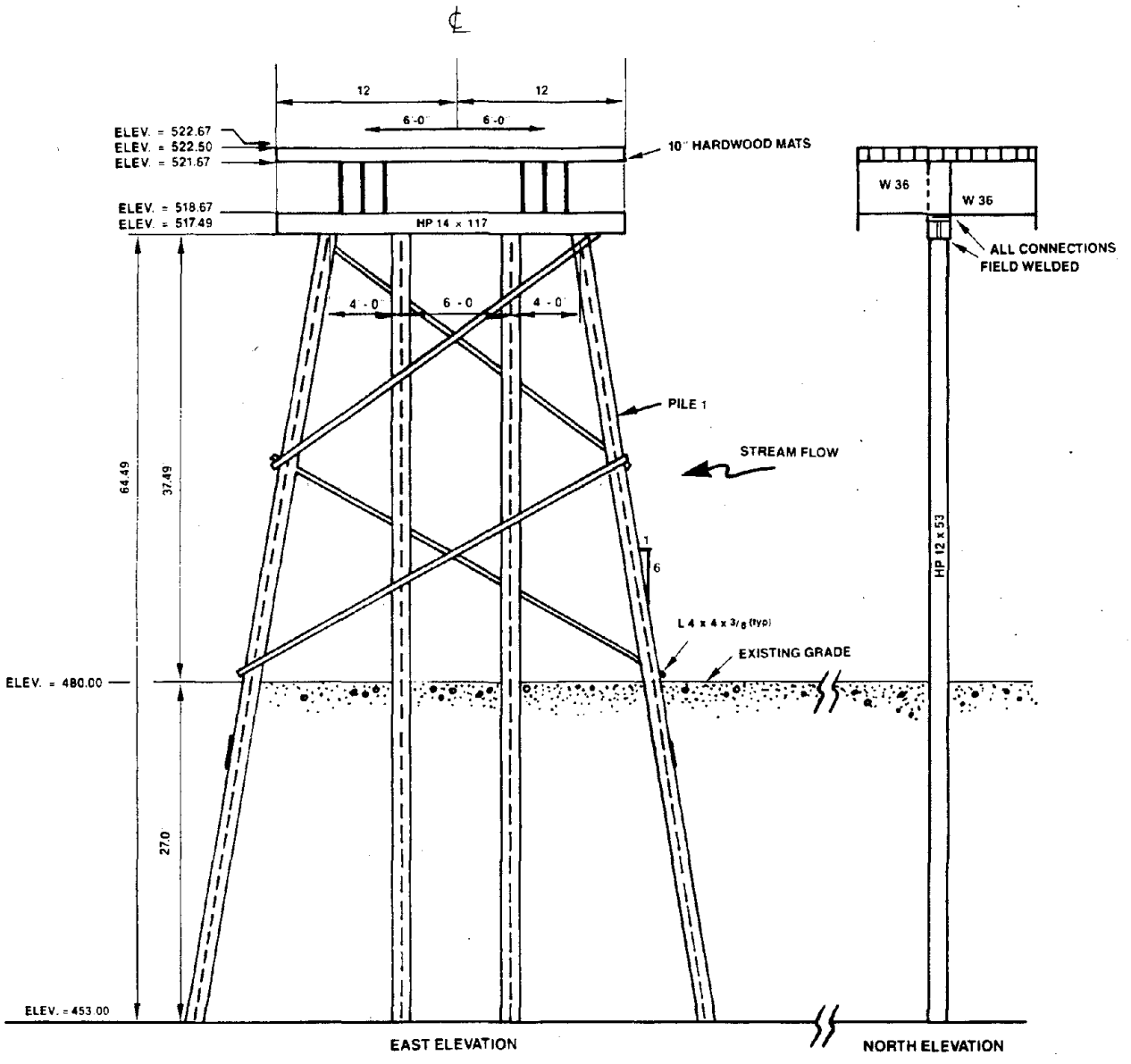
CONSULTANT'S DESIGN, TEMPORARY BRIDGE, ELEVATION



CONTRACTOR'S DESIGN, TEMPORARY BRIDGE, ELEVATION

(2) TWO GIRDERS
(4) FOUR GIRDERS
* AS-BUILT SECTION WAS W36×170

Figure 6. - - Side view schematic of temporary bridge designs.



CONTRACTOR'S DESIGN, TEMPORARY BRIDGE, PILE BENT 2

Figure 7.--Front view of NECC's pile bent 2 as designed.

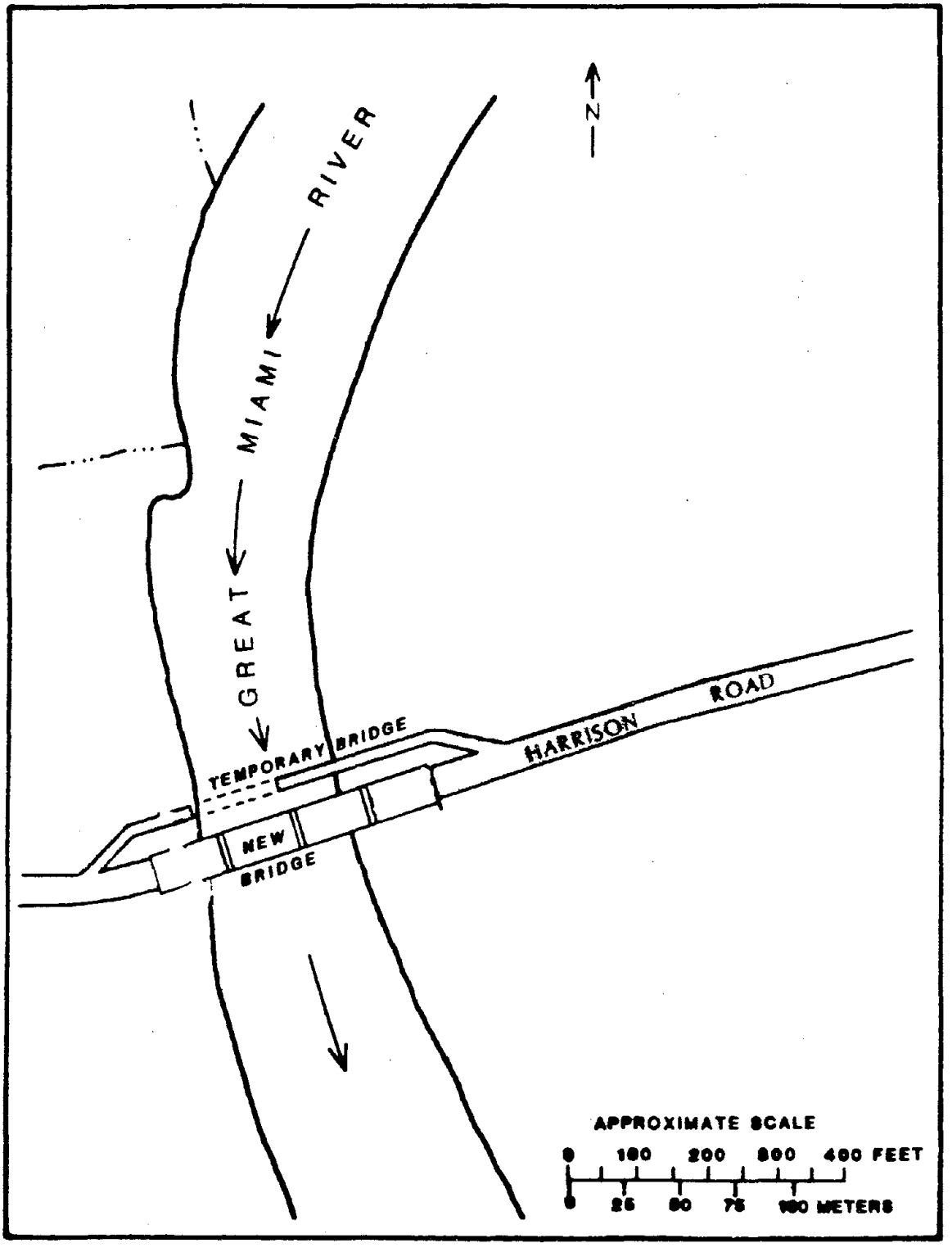
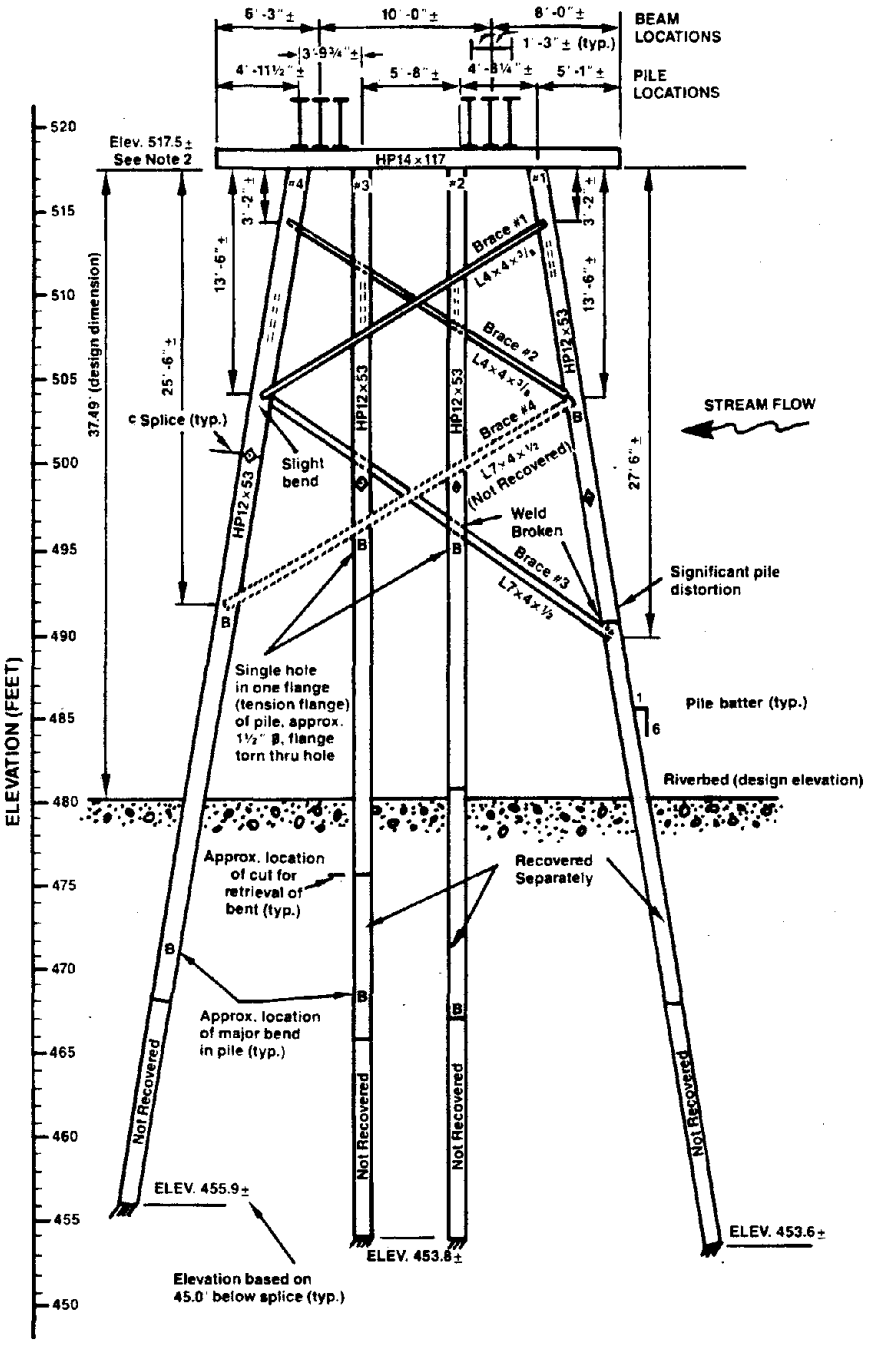


Figure 8.--Plan view schematic of the temporary bridge over the Great Miami River.



EAST ELEVATION OF PILE BENT 2 (As Constructed)

Notes: 1) This drawing based on measurements of salvaged materials.
2) Pier cap elevation based on construction plans.
3) B = Location of bend

Figure 9.--East elevation of NECC's pile bent 2 as constructed.

wedged to the top flange of the W-beams with bolts and angle irons. W-beam guardrails spanned the bridge on both sides of the asphalt lane.

Bridge Wreckage and Documentation.--On June 9, 10, and 11, 1989, the bridge wreckage was documented as it was found on the riverbed. The four 36 x 170 W-beams of span 2 were found near the west bank of the river. One beam rested on the riverbed upstream, away from the other beams. The two 36 x 230 W-beams of span 3 were found near the second pier of the replacement bridge. The remaining sections of each span were still joined together and were partially buried in the loose granular riverbed material. (See figure 10.)

Pile bent 2 was found approximately where it had been constructed. The pile cap was facing and parallel to the west bank, and the four piles were bent with the bottom ends still buried in the riverbed. The pile cap was undamaged, and all welds between the piles and pile cap were intact, as were the welds on the upper cross bracing. The lower brace on the east face fell off as the bent was removed, and the brace was never recovered. The brace on the west face remained welded to piles 3 and 4 (piles numbered from upstream to downstream). However, the welds connecting the west brace to piles 1 and 2 were fractured. (See figure 11.) Trees and branches were wedged within and on top of the piles. The pile cap was 1 to 2 feet above the riverbed, and the splice plates[15] of each pile were intact.

The second and third piles were parallel to each other and about 1 foot apart, but twisted so that the web of one of the piles was parallel to the riverbed. The flange was slightly bent where the third pile entered the riverbed. The fourth pile was found about 3 to 4 feet above the riverbed and bent acutely so that it entered the riverbed in an almost perpendicular fashion. The flange was torn at the pointed bend.

The pile bent was removed from the river and inspected on June 12, 1989. Two of the piles were bent approximately 90 degrees, the pile located farthest upstream (pile 1) was bent approximately 50 degrees, and the pile located farthest downstream (pile 4) was bent slightly more than 90 degrees. Burn holes[16] and tears in the flange were noted on piles 2 and 3 slightly below the area where the lower cross bracing was attached.

Bridge Design Standards

AASHTO Guidelines.--The AASHTO "Standard Specifications for Highway Bridges" are intended to serve as a standard or guide for the preparation of State specifications and for reference by bridge engineers. The AASHTO specifications are referenced in the Federal Highway Administration (FHWA) Federal Aid Program Manual 6-2-1-1, dated September 1987 and entitled "Design Standards for Highways," for use on Federal-aid highway projects. The State of Ohio has adopted these standard specifications and uses them in conjunction with additional State specifications.

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[15]A splice plate is a piece of metal welded to a structural element to cover a joint to provide strength and stability.

[16]Holes cut into the pile flange with a torch during construction to aid in placing the piles.

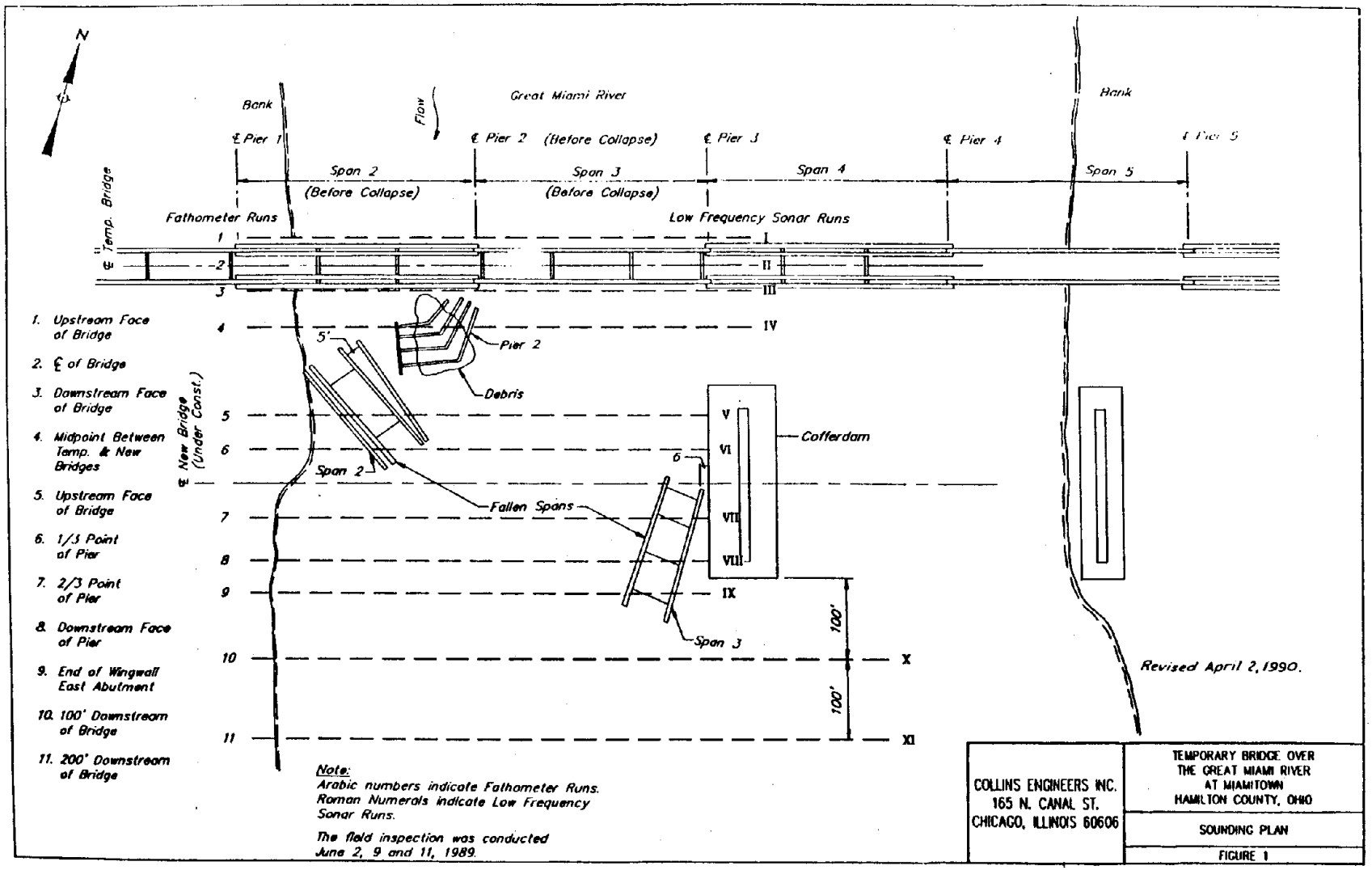


Figure 10.--Schematic of bridge wreckage.



Figure 11.--Pile bent 2 after it was retrieved from the river.

The AASHTO specifications do not differentiate between permanent bridges and temporary bridges; consequently, there are no separate criteria specifically relating to temporary bridges. Sections of the AASHTO specifications that govern the design of bridge structures address dead loads, live loads, impact loads, longitudinal forces, centrifugal forces, wind loads, stream current, floating ice, drift (debris), and additional forces under particular conditions.

A bridge may be subjected to several of the above mentioned loads at the same time and the AASHTO specifications therefore establish combinations of loads (group loading criteria) that can be considered as acting on a structure simultaneously. By using combination loading, only a percentage of the sum of the maximum effects is considered, since the likelihood of the peak value of these events occurring simultaneously is remote.

> AASHTO specification section 3.7.4 (Minimum Loading) states:
>
> Bridges supporting Interstate highways or other highways which carry, or which may carry, heavy truck traffic shall be designed for HS20-44 Loading....
>
> AASHTO specification section 3.15 (Wind Loads) states:
>
> A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure. For girders and beams. . . 50 pounds per square foot.
>
> The total force shall not be less than 300 pounds per linear foot on girder spans.
>
> AASHTO specification section 3.18 (Force from Stream Current, Floating Ice and Drift) states:
>
> All piers and other portions of structures that are subject to the force of flowing water, floating ice, or drift shall be designed to resist the maximum stresses induced thereby.
>
> AASHTO specification section 3.22 (Combination of Loads) states:
>
> Each component of the structure. . . shall be proportioned to withstand safely all group combinations of. . . forces that are applicable to the particular site or type [of structure].
>
> AASHTO specification section 1.3 also states that:
>
> Hydraulic studies of bridge sites are a necessary part of the preliminary design of a bridge and should include (a) maps, stream cross sections, aerial photographs, (b) complete data on existing bridges, including dates of construction and performance during past floods, (c) available high water marks with dates of occurrence, (d) information on ice, debris, and channel stability, (e) factors affecting water stages such as high water from other streams, reservoirs,

> flood control projects, and tides. Hydrologic analysis should include (a) flood data applicable to estimating floods at site, including both historical floods and maximum floods of record, (b) flood-frequency curve for site, (c) distribution of flow and velocities at site for flood discharges to be considered in design of structure, (d) stage-discharge curve for site. Hydraulic analysis should include (a) backwater and mean velocities at bridge opening for various trial bridge lengths and selected discharges, (b) estimated scour depth at piers and abutments of proposed structure.

ODOT Requirements.--The ODOT has its own bridge design manual that gives detailed instructions regarding bridge plan preparations as practiced in ODOT's Bureau of Bridges. Directions, provisions, and requirements for construction and materials are promulgated in ODOT's "Construction and Materials Specifications." Although this publication is not a design manual, item 502.02 in the 1987 edition addresses temporary structures and states in part:

> The temporary structure shall have an adequate waterway opening, but generally not less than 75 percent of the effective waterway opening of the proposed structure or may be designed to provide for a five-year frequency flood as a minimum. The contractor shall be responsible, however, for any damages caused by upstream flooding due to insufficient temporary structure size or the accumulation of debris or sediment in the channel. . . .
>
> The temporary structure shall be designed for a loading of HS20-44 at unit stresses not to exceed 136.5 percent of those specified by pertinent sections of AASHTO "Standard Specifications for Highway Bridges."

The chairman of the AASHTO Highway Subcommittee on Bridges and Structures testified at the public hearing that he interprets "pertinent sections" to mean that the designer should reference the type of material to be used for the structure. He stated:

> . . .if you have a steel structure, you use the portions of the specifications relative to steel structures. If you have a concrete structure, you use those, and not try to apply timber and so on and so forth. If you're dealing in a structure over waterways, you deal with the items that deal with waterways. If it's over a highway, then you deal with those. To me, that's what pertinent means.

He also stated that "pertinent sections" refers to those sections dealing with loads and stresses, including vertical loads (dead, live, and impact loads) and lateral loads (wind, stream flow, ice, and debris loads).

No Federal or State funds were used in the design and construction of either the temporary or the permanent bridges. Therefore, there was no State or Federal review of the design plans. However, the county was required to submit the plans to the State for review, according to an Ohio law (Ohio Revised Code, Section 5543) that states:

> No contract for the construction or repair of a bridge, the entire cost of which construction or repair exceeds fifty thousand dollars, shall be entered into by the county unless the plans are first approved by the state director of transportation.

The HCE's office did not submit any design plans to ODOT for review. According to testimony from an ODOT bridge engineer, of 88 counties in the State of Ohio, about 20 comply with this law. He also testified that had the bridge design been submitted for review, the ODOT would have reviewed the temporary bridge plans for structural adequacy, including adequacy for lateral loads, wind loads, and impact loads. He stated that if the design plans did not include provisions for lateral loading, ODOT would not have approved the plans.

Hamilton County Guidelines.--HCE representatives stated that they adhere to the ODOT Bridge Design Manual and "Construction and Materials Specifications" when designing various projects, and the ODOT specifications were referenced in the contracts with the consultant. The HCE representative testified at the Safety Board's public hearing that his office did not submit the plans for approval as required by the Ohio Revised Code, Section 5543, because it did not have the plans available to send to ODOT for prior approval when the contractor submitted the alternate bid and because it was concerned about the time consumed by the process of sending plans to ODOT.

Geological Information

The Great Miami River Valley has a 3/4-mile-wide flood plain surrounding the Great Miami River. The Great Miami River and its tributaries drain more than 3,900 square miles across southwest Ohio. (See figures 12 and 13.) In the vicinity of the Harrison Road bridge, the soil is glacial till underlain by sand, gravel, and cobbles and, in some areas, clay layers. The bedrock is soft shale.

Meteorological Information

On May 26, 1989, the Cincinnati weather was mostly cloudy with a visibility of 20 miles. The temperature was 80 degrees Fahrenheit with a westerly wind of 10 mph. In Miamitown, witnesses stated that when the bridge collapsed, skies were clear and sunny and the temperature was about 80 degrees Fahrenheit.

Rainfall.--In the Great Miami River basin on May 26, 1989, rainfall ranged from a high of 3.37 inches to a low of 0.78 inches. Many of the water basins that drain into the Great Miami River had received rainfall of approximately equal or greater magnitude 3 days earlier. Rainfall data records indicate that most of the rain fell on May 26, 1989, between 3 a.m. and 5 a.m. The average total amount of rainfall in the 24-hour period in the Miami Conservancy District (MCD)[17] amounted to 2.08 inches. According to the National Weather Service (NWS), this amount had an approximate recurrence interval of about 2 years. Daily rainfall records are collected at more than

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[17]A public corporation that was formed in 1915 to protect the major cities along the Great Miami River from floods. After the 1913 flood, during which 300 people died and more than $100 million of damage occurred, the Ohio legislature passed the Ohio Conservancy Act, which allowed for the creation of conservancy districts. Hamilton County is not included in the MCD.

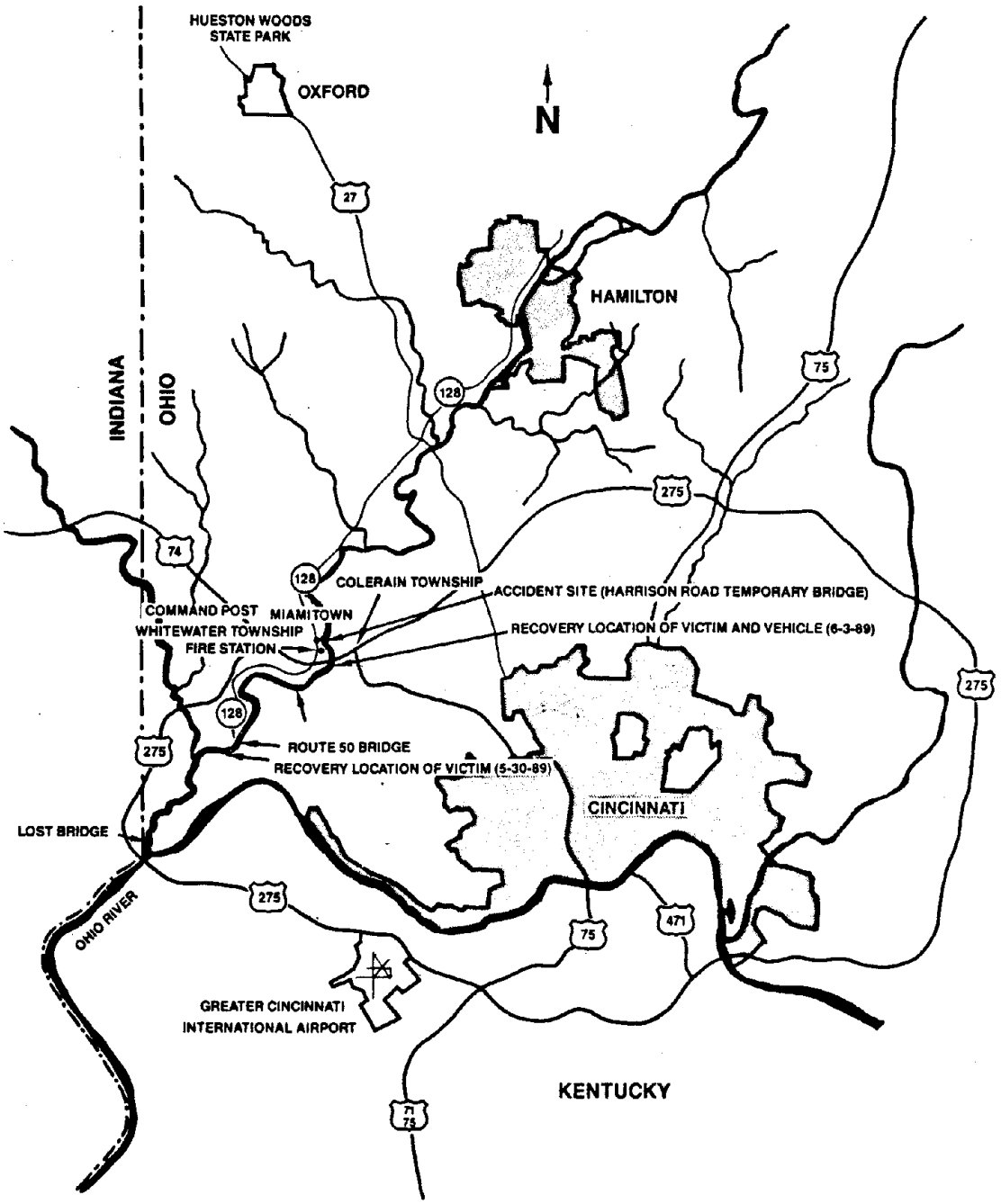


Figure 12. --Location of Miamitown, Ohio.

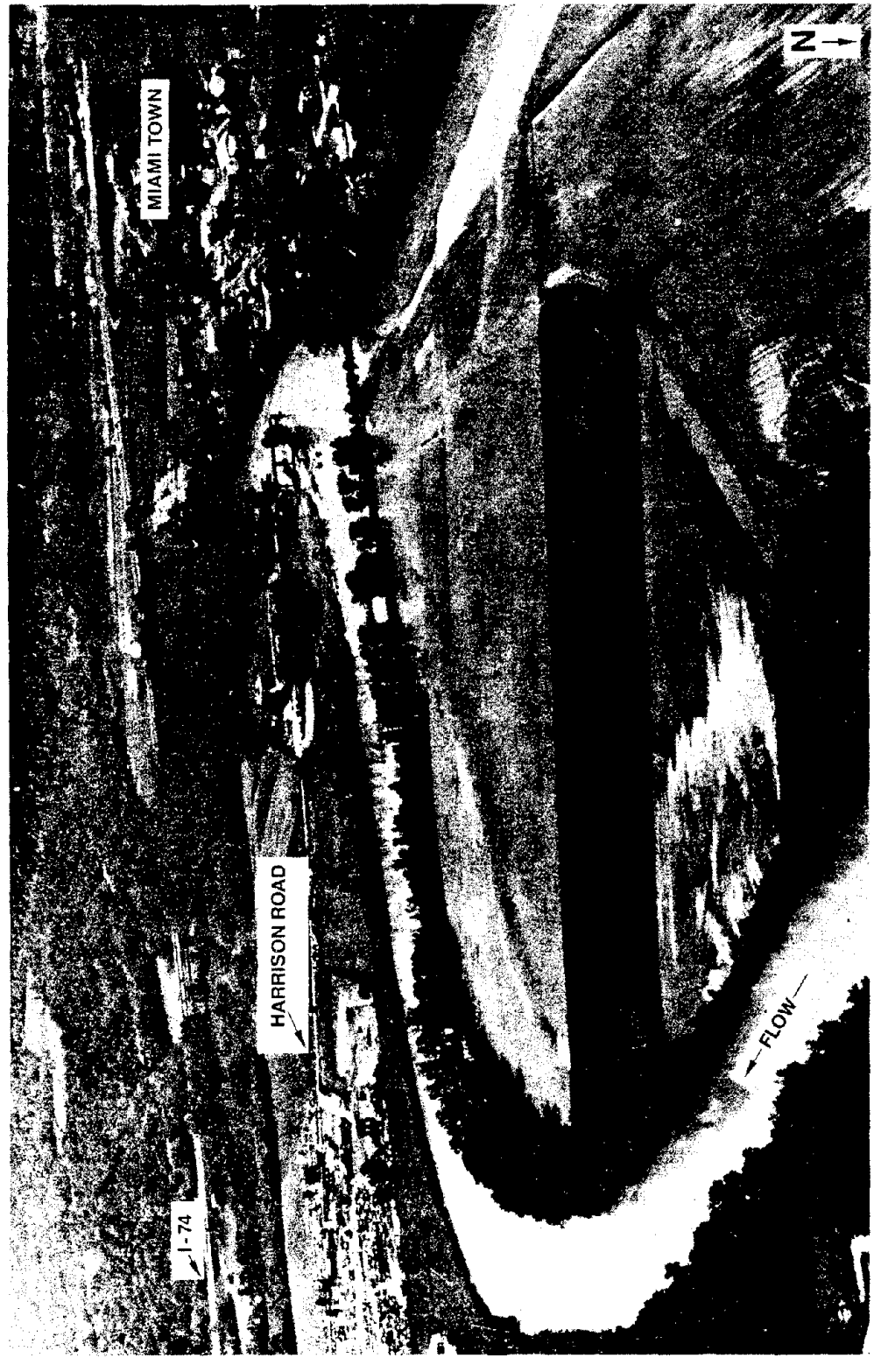


Figure 13.--The Great Miami flood plain.

40 stations, and stream flow measurements are recorded at 30 locations by MCD staff.

Flood Warning/Flood Watch.--On Thursday, May 25, 1989, at 3:48 p.m., the NWS issued a flood watch for all but extreme northern and northeastern Ohio for that night and the next day. A flood watch is issued when a flood threat exists in the areas indicated, and the purpose of the watch is to advise citizens to keep informed and to be ready for quick action if flooding is observed or a warning is issued. This watch included Hamilton County, as well as the surrounding counties in the southwest part of the State. Flood watches and warnings are disseminated on the National Oceanographic and Atmospheric Administration Weather Wire, the National Distribution Circuit to the Family of Services, and the Ohio Highway Law Enforcement Automated Data System circuit. The flood watch on May 25 stated:

> A large complex system of thunderstorms will be moving through Ohio tonight and Friday. Two to four inches of rain are possible in the Watch Area by Friday evening. Many rivers and streams in the Watch Area are already above normal levels and this additional rain may cause them to flood.

On Friday, May 26, 1989, the NWS issued several flood warnings for the Great Miami River. The 11:30 a.m. and 2:20 p.m. flood warnings predicted the river would rise to 20 feet; the 5:20 p.m. warning predicted 23 feet; and the 9:40 p.m. warning predicted 24 feet by Saturday morning, May 27. The flood warnings advised that a heavier concentration of rain was falling north of Hamilton, Ohio, and that the flooding of the Great Miami River would continue.

River Gauge Readings.--Water level readings obtained from a person monitoring the river gauge located at the Harrison Road temporary bridge were reported to the Ohio River Forecast Center. This river gauge had been installed on pile bent 6 on the east end of the bridge. The gauge is numbered from 13 feet (elevation 495.15 feet) to 33 feet (elevation 515.15 feet). At this location the flood stage is considered to be 16 feet (elevation 498.15 feet). Readings from this gauge taken shortly before and after the temporary bridge collapse about 5:25 p.m. were used to estimate the flood stage at the time of the collapse. The acting hydrologist-in-charge at the RFC stated that the sheriff's office was notified of the potential for flooding at 11:34 a.m. on May 26. The hydrologist said that a record flood of about 34 feet was recorded in Miamitown in 1959.

Hydrology Information

The Safety Board requested that the U.S. Geological Survey (USGS) gather information and evaluate the hydrologic and hydraulic conditions that existed during the May 26 flood and bridge collapse. One week after the collapse, from May 31 through June 2, the USGS measured the high water marks on both sides of the river along a reach extending approximately 1,000 feet upstream and downstream of the Harrison Road bridge site. Approximately 500 feet upstream and 500 feet downstream of the bridge site, high-water marks indicated water-surface elevations of about 507.0 feet and 503.4 feet, respectively.

The data from two MCD discharge measurement gauges (the Hamilton and Indian Creek gauges, which were approximately 15 and 19 miles, respectively, upstream from the Harrison Road bridge) were used to estimate the conditions at

the bridge site. A hydrograph[18] of the river reach at the bridge was similar to a hydrograph of the river reach in the area of the gauges; therefore, the USGS was able to estimate conditions at the bridge site. The discharge at the time of the bridge collapse was about 46,700 cubic feet per second (cfs), and the peak discharge at 1 a.m. on May 27 was 67,000 cfs. The May 26 peak discharge of 64,600 cfs at the Hamilton gauging station has been exceeded in magnitude (on an annual basis) eight times during the period from 1922 to 1988, most recently in 1968.

USGS-gathered data, including water discharge, water-surface elevations, and riverbed material roughness coefficients, were used to estimate water velocities. The velocities at pile bent 2 for the preflood and postflood geometries were estimated to have been 10.7 and 9.2 fps, respectively, for the peak discharge of 67,000 cfs. The velocities at pile bent 2 for the preflood and postflood geometries were estimated to have been 9.1 and 7.6 fps, respectively, for the discharge estimated to have occurred at the time of the bridge collapse, 46,700 cfs. Return periods[19] indicate that the flood during the time of collapse, 46,700 cfs, would have a recurrence frequency of about 2 to 3 years and that the peak flood, 67,000 cfs, would have a recurrence frequency of about 9 years. The following table lists flood frequencies estimated from the flood frequency curve for the Hamilton County gauging station.[20]

Table 1. Flood Frequency for Hamilton Gauging Station (1922-1988)
NTSB Estimate from USGS

Annual Exceedance Probability	Flood Frequency Curve (cfs)
0.50 (2 years)	39,000
0.20 (5 years)	57,000
0.10 (10 years)	68,000
0.02 (50 years)	87,500

Riverbed Elevations

Construction of the Temporary Bridge.--In June 1987, a consultant (CDS Associates) for the HCE's office surveyed the riverbed at the old truss bridge and found that in the 28-foot-wide channel, at a point 50 feet downstream of pile bent 2 of the temporary bridge, the riverbed elevation was 470.5 feet. In July 1987, another consultant (H.C. Nutting Co.) to the HCE conducted a geotechnical investigation of the site. Seven soil boring samples[21] were taken at the existing truss bridge abutments, and three boring samples were taken at the proposed permanent bridge pier locations. Only one boring hole was drilled in the riverbed, approximately 30 feet from the west bank and 50 feet downstream of the proposed temporary bridge. The water level was at an elevation of 483 feet; concrete rubble was encountered at an elevation of 478 feet. Medium-dense sand and gravel were

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[18]The characteristics of the response of the stream during a storm event.

[19]The average interval during which a flood of a given size is exceeded as an annual maximum.

[20]Derived by the USGS following the Interagency Advisory Committee on Water Data Bulletin 17B entitled "Guidelines for Determining Flood Flow Frequency" published by the U.S. Department of Interior.

[21]Measured cylindral samples of soil extracted from the riverbed by means of a drilling tool.

encountered at elevation 475, very soft shale bedrock was encountered at elevation 457, and the boring was terminated at 97 blows per 6 inches at elevation 453.

NECC's temporary bridge design plans, dated September 1988, showed that the riverbed elevation at pile bent 2 was 480 feet and that the lowest riverbed elevation was 472.5 feet at a point 40 feet west of pile bent 2. The water surface elevation was shown on the plans as 484.4 feet. A construction platform (a finger road constructed of gravel dredged from the riverbed) was placed from the east bank downstream of the temporary bridge in late October 1988 to drive the piles on both the temporary and permanent bridges. The superintendent testified that the platform was 5 or 6 feet deep, which would be at an elevation of 485 or 486 feet based on the riverbed elevation of 480 feet listed on NECC's design plans. Neither the HCE project engineer nor the NECC recorded the number of blows required to drive the piles of pile bent 2 into the ground.

Postcollapse.--In June 1989 (after the collapse), a consultant hired by HCE surveyed the riverbed elevations. He found that the riverbed elevation on the upstream side of pile bent 2 was 471.5 feet and that the riverbed elevation on the downstream side of pile bent 2 was 471 feet. The lowest riverbed elevation on the upstream side was 468.5 feet (30 feet west of pile bent 2), and on the downstream side the lowest elevation was 471 feet (85 feet west of pile bent 2). In July 1989, the consultant also measured the lengths of the recovered piles, and the Safety Board used those measurements to calculate the following riverbed elevations for each of the piles of pile bent 2 (numbered from upstream to downstream beginning with pile 1); the piles were cut off at the riverbed:

Pile 1 - 469 feet
Pile 2 - 468 feet
Pile 3 - 466 feet
Pile 4 - 468 feet

Tests and Research

Metallurgical Information.--A Safety Board metallurgist examined the bridge wreckage from pile bent 2 on the river bank on June 19, 1989. Four cross braces had been welded to the flanges of each of the piles. The cross braces had been attached directly to the flanges of each pile by welds running along the edges of the braces. In most cases, the welds ran continuously across the flange face along one edge of the brace, and intermittent welds ran along the other edge of the brace contacting the flange face. The portion of pile 1 where brace 3 had been attached was recovered after the Safety Board's initial wreckage examination. An examination of this pile by a consultant for HCE indicated that it had not bent at this location but that distortion was significant.

Of the 16 cross brace weld attachment areas on the 4 piles, 6 areas were found to have separated. At these 6 locations the welds had fractured either directly through the weld fusion zones or immediately adjacent to the fusion zones. Very little weld penetration into the base metal was noted on those welds that broke along the edge of the fusion zone. In the immediate area of each of these separations, local deformation in the form of buckling, twisting, or bending was noted in either the pile or the brace or both. A significant amount of local deformation was noted in the members at several of the intact weld locations.

A 13.5-inch segment from the lower end of the lower west face cross brace was flame cut and removed for examination. This cross brace section was found to be cracked along a line of transverse welds 10.25 inches from the lower end.

Underwater Inspection of Wreckage and Riverbed.--In June 1989, Collins Engineers, Inc. (CEI), under contract with the FHWA and the HCE, performed an underwater inspection of the submerged portions of the bridge wreckage and took soundings of the riverbed. The main purpose of this work was to determine the configuration and location of the submerged bridge components and to determine the condition of the riverbed. Soundings were taken along lines approximately parallel to the centerline of the temporary bridge from the upstream face to approximately 200 feet downstream of the temporary bridge. The soundings were made with a recording-type fathometer using a 192-KHz, 20-degree transducer. Low-frequency sonar, with a 14-KHz tuned transducer, was used to produce subbottom profiles to determine whether there were any scour holes, whether infilling of scour holes had occurred at the site, and whether there were any differences in the density of the riverbed material.

Sounding data just upstream and downstream of the collapsed temporary bridge provided evidence of infill material[22] in the low-water channel (approximately 30 feet west of pile bent 2), as well as infill material to the east side of pile bent 3. Infilling was not apparent in the vicinity of pile bent 2. Neither CEI nor the USGS were able to determine from this data whether scour at pile bent 2 had occurred.

FHWA Engineering Analysis of Collapsed Bent.--The FHWA performed a limited linear two-dimensional frame analysis of pile bent 2, using a computer program called STRUDL[23] to determine the loading conditions that would precipitate a failure of a primary structural member in pile bent 2. A computer model was developed to simulate the pile bent structure as it was designed and as it was constructed. The secant formula[24] was used to determine stresses in the piles.

The analysis by FHWA showed that the substructure failure could have occurred when pile bent 2 was subjected to the forces of dead load, a live load of one passenger car, stream flow, and a debris force of 5 tons. Under the 5-ton debris load, pile 4 could have failed from compression flange buckling in flexure, general column buckling from axial loads, or buckling from combined flexure and axial loads.[25] As the bent was displaced sideways by the debris and stream flow forces,

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[22]After a flood, when river velocities decrease, material is redeposited in scour holes. This is referred to as "infilling." Since infill material often has a different density than the adjacent unscoured material, scour can be measured by determining the interface where the density change occurs.

[23]STRUDL is an acronym for Structural Design Language; it is a series of computer programs for solving problems in structural engineering and was used in this accident to perform a linear elastic, static analysis of pile bent 2.

[24]The secant formula is one of many equations available for structural analysis that is used to determine the safe load with eccentric loading parallel to the axis of the column. Its use in forensics in this collapse is limited to determining the stresses at yield.

[25]Buckling, which is the kinking, wrinkling, bulging, or otherwise loss of shape of a structure, occurs when loads exceed the structural strength of the member.

moments in all four piles would have been increased dramatically, thus increasing their susceptibility to buckling failure.

NECC Engineering Analysis of Collapsed Bent.--On April 6, 1990, NECC submitted a structural analysis of pile bent 2 that was performed using the computer program EZFEAT[26] and the interaction formula.[27] The NECC analysis indicated that if the bracing remained attached, a lateral force from stream flow and debris of about 20.5 tons would cause pile 4 to fail and that if the lower west face brace failed first, a lateral force from stream flow and debris of about 13.5 tons would cause pile 4 to fail.

At the request of the Safety Board, the FHWA performed further analyses. The calculations with fixity at 471 feet indicated that a lateral force from debris of about 11 tons would cause pile 2 to fail from lateral buckling if the lower east face brace failed first and that a lateral force from debris of about 13 tons could have caused a lateral buckling failure in pile 4 with all the bracing attached.

In addition, the FHWA concluded that the yielding of pile bent 2 could have occurred at debris loads as low as 5 tons based on the secant formula analysis with or without the bracing intact. Moreover, according to FHWA, an interaction analysis (combined column and axial loads and flexure) indicated that allowable design stresses are exceeded at debris loads as low as 5 tons without the bracing.

University of Maryland Engineering Analysis of Collapsed Bent.--In May 1990, the Safety Board contracted with the University of Maryland (UMD) to conduct structural calculations to determine the lateral load capacity of the collapsed structure and the ability of the bridge to meet AASHTO lateral load specifications. Two types of computer analysis were carried out to determine the lateral load capacity and the sequence of failure from both elastic buckling[28] and an elasto-plastic[29] type failure.

The UMD concluded that the results of the elastic buckling analysis did not correspond with the physical evidence and that an elasto-plastic failure (or formation of plastic hinges[30] on the H-piles) was the probable mode of failure. The UMD calculated that the formation of plastic hinges began at a lateral debris load of about 7.5 tons and that overall collapse would occur between 11 and 12.5 tons. These loads assumed an initial failure at the connection between the lower west face brace and pile 1. The UMD estimated that had the connection not failed, the bent could have withstood a lateral load in excess of 15 tons. In addition, the most

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[26]EZFEAT is an acronym for Easy Finite Element Analysis; it is a series of computer steps for solving problems in structural engineering and was used in this accident to perform a linear elastic, static frame analysis of pile bent 2.

[27]A method of approximating the ultimate strength when a structural member is subjected to combined loading, such as bending and axial compression.

[28]Elastic buckling analysis is a method used to determine the upper bound of the load-carrying capacity of a structure before buckling occurs.

[29]Elasto-plastic analysis is a method used to determine the upper bound of the load-carrying capacity of a structure before plastic deformation occurs.

[30]A plastic hinge occurs when the loads on a structure exceed the elastic limit, causing deformation of the structure.

probable characterization of the debris that actually struck the pile bent and caused it to fail was a combination of impact and accumulated debris forces. The UMD analysis also indicated that an object, such as a 50-foot, 7- to 14-inch-diameter floating log, striking the pile bent head-on at a stream flow velocity of 9 fps, was capable of providing the lateral impact force required for collapse of the pile bent.

AASHTO specifications require a safety factor of 2.12 when designing long columns used in bridge structures. The UMD applied this safety factor to the NECC temporary bridge design. Using the results of the elasto-plastic analysis, the UMD calculated that the NECC temporary bridge could sustain a safe lateral load of about 3.5 tons. The UMD also calculated that the NECC temporary bridge would had to have been designed to withstand a safe lateral load of about 23 tons in order to conform with AASHTO's group loading specifications for stream flow, ice, and wind.

In August 1990, the Safety Board asked the UMD to conduct an engineering analysis of the GOP temporary bridge design using the elasto-plastic analysis method to determine the bridge's lateral load capacity and its ability to meet AASHTO group loading criteria. The UMD concluded that the GOP-designed bridge would experience an initial formation of plastic hinges at a lateral debris load of about 39 tons. According to the UMD estimate, a lateral load greater than 65 tons[31] would have caused the overall collapse of the structure. The analysis indicated that the GOP design would have withstood the calculated lateral loads that collapsed the NECC bridge and that the GOP design met the AASHTO provisions for group loadings.

Survey of Temporary Bridges

On August 22, 1989, the Safety Board sent letters to 56 States and territories and to 1,100 of the 3,042 counties in the United States asking for information concerning the number of permanent and temporary bridges within those jurisdictions, whether the design criteria for temporary bridges differ from those specified for permanent bridges, and if so, in what way.

The Safety Board received responses from 44 States, which reported 1,037 temporary bridges[32] representing 92 percent of the bridges in the National Bridge Inventory (NBI).[33] Of the 44 responding States, 14 reported no temporary bridges, 18 reported 1 to 9 temporary bridges, 8 reported 10 to 100 temporary bridges, and 4 reported more than 100 temporary bridges.

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[31]The high degree of redundancy, the resistance to lateral loading of the GOP design, the complex interconnection of cable elements, and the three-dimensional behavior of the bent necessitated computer time beyond the scope of the project. Thus, the UMD did not calculate the value of the collapse load.

[32]The 1988 FHWA designation "temporary structure(s)" applies to those at which conditions require measures to facilitate traffic flow. It is used (1) when bridges are shored up, (2) when temporary repairs are made to keep a bridge open, and (3) when temporary structures, temporary runarounds or bypasses, and other temporary measures, such as barricaded traffic lanes, are in place to keep the bridge open.

[33]As of June 30, 1988, there were 577,710 bridges in the FHWA-maintained NBI that includes 50 States, Puerto Rico, and the District of Columbia. Of these bridges, 274,583 are on the Federal-aid system and 303,127 are not.

Of the 1,100 local jurisdictions, responses were received from 645 counties, reporting 97,510 permanent bridges (16 percent of those in the NBI) and 105 temporary bridges. Of the 35 States that were represented by counties responding, 17 reported no temporary bridges. Some temporary bridges were reported by both counties and States.

Many States and counties reported that they used Acrow or Bailey bridges as temporary bridges. Both are prefabricated modular steel panel bridge spans that can be used as replacement spans on existing supports. The majority of the States and counties responding to the survey question regarding design criteria indicated that they used the same standards for permanent and temporary bridges. Those using different criteria cited "legal loads," "10-ton loading," and "HS-15" and "HS-20" loading as the major criteria to which temporary bridges would be designed. Some indicated that geometric standards and hydraulic criteria for temporary bridges would be less stringent than those for permanent bridges.

Most respondents cited AASHTO or State Department of Transportation specifications as the criteria used when constructing a temporary structure. Other respondents indicated that they do not use temporary bridges but close the route until permanent replacement bridges are constructed or until repairs have been made to the existing structure.

ANALYSIS

General

The performance of the passenger car driver before the collapse did not affect the outcome of this accident. The span on which the driver was traveling collapsed before she had an opportunity to make an evasive maneuver. The passenger car driver directly behind the car that fell into the river was attentive and reacted appropriately. She immediately began backing her car off the bridge and motioned other drivers behind her to back up. Another passenger car driver, traveling in front of the car that fell into the river, also was attentive. She initially thought her tires were going flat because her car was swaying from side to side. She accelerated to get off the bridge, looked in her rearview mirror, and saw the blue passenger car fall into the river. Although seven witnesses stated that they saw a pickup truck fall into the river, the HCSO never found a pickup truck and has no reports of missing persons that are thought to be related to the bridge collapse.

The initial call about the collapse was received by the Hamilton County Communications Center at 5:27 p.m. Three minutes later, a sheriff's deputy and the fire chief arrived at the scene of the collapse. Search and rescue efforts were seriously hampered because of the rapid river flow, depth of the river, and debris in it. Witnesses at the site sought to locate the vehicles or their occupants in the river but were unsuccessful. The Safety Board concludes that the response of emergency personnel was timely and that their actions were appropriate under these conditions.

Failure Mode Sequence

Superstructure.--Based on all available evidence, the Safety Board concludes that no preexisting defects in the superstructure caused the bridge collapse. There were no cracks, physical deformities, or misalignments reported by either the HCE bridge engineer or the NECC superintendent who observed the superstructure just

before the collapse. A postaccident inspection of the bridge wreckage did not reveal any superstructure defects. Furthermore, based on the witnesses' descriptions of the vehicles on the bridge, the superstructure was not overloaded at the time of the collapse. The physical evidence and witnesses' statements indicate that the spans and the pile bent fell rapidly during the collapse sequence, suggesting that failure of a substructure element may have initiated the collapse.

Substructure.--Examination of the bridge wreckage following the collapse revealed that the welds connecting the cross bracing to the piles of pile bent 2 were of poor quality. However, the Safety Board metallurgical examination indicated, and the UMD analysis concurred, that the welds connecting the lower east brace to piles 1, 2, 3, and 4 failed during the collapse and not because of preexisting defects. Results of the NECC and UMD analyses showed that the weld connecting the lower brace to the west flange on pile 1 failed before the collapse. The NECC and UMD analyses also indicated that the failure of this weld substantially reduced the pile bent's resistance to lateral loads. However, the cross bracing members were not designed to accommodate lateral loads; they were used to reduce the effective length of the columns and thus increase the vertical load-carrying capacity of the bridge.

Examination of the bridge wreckage revealed burn holes in the west flanges of piles 2 and 3 just below the lower bracing. The burn holes slightly reduced the strength of each affected pile and created potential stress raisers.[34] Although plastic hinges began to form in the piles at the bracing points, bending occurred at the burn holes just below because of the reduced section. Nonetheless, bending would have occurred in the absence of burn holes, which are unlikely to have significantly reduced the lateral load-carrying capacity of the pile bent. The Safety Board concludes that the burn holes did not significantly reduce the lateral load capacity of the bridge.

The FHWA, NECC, and the UMD performed independent engineering analyses to determine the most likely mode of failure and the magnitude of debris loading needed to precipitate a failure of pile bent 2. The FHWA and NECC assumed that failure may have resulted from elastic buckling and performed further analyses to determine the loading conditions that would precipitate a structural failure of a member in the pile bent. Although the FHWA and NECC used different methodologies to analyze the structure and to determine the magnitude of debris loads needed to cause a failure, they both concluded that a buckling failure of the piles had occurred, causing pile bent 2 to collapse. Nonetheless, after performing the elastic buckling analysis, the UMD concluded that this type of failure could not explain the formation of the plastic hinges indicated by the physical evidence.

In performing its analysis, the UMD assumed that the pile bent failure resulted from some type of elasto-plastic yielding that led to the formation of plastic hinges. The UMD analysis indicated that at a combined impact and accumulated debris load of 7.5 tons, plastic hinges would begin to form. The UMD also concluded that collapse would occur when a critical number of plastic hinges[35] had developed

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[34]Stress raisers are changes in contour or discontinuities in structure that cause local increases in stress.

[35]When the number of plastic hinges formed exceeds those required for elastic stability, the overall collapse of the pile bent occurs.

throughout the substructure, at a combined impact and accumulated debris loading between 11 and 12.5 tons. Based on UMD's engineering analysis and the physical evidence, the Safety Board concludes that the collapse of pile bent 2 resulted from the formation of plastic hinges due to a combination of impact and accumulated debris loading on the upstream side of the pile bent.

Adequacy of the Bridge Design

The Safety Board's review of the NECC temporary bridge design indicates that the bridge was designed in accordance with AASHTO (HS20-44) vertical loading and with the ODOT waterway opening specifications for temporary structures. At the Safety Board's request, the UMD reviewed the NECC temporary bridge to determine whether it would have met AASHTO specifications for group loading combinations and found that it did not. According to the UMD report, if the NECC temporary bridge had been designed to conform with AASHTO lateral load specifications, pile bent 2 would have had a safe load capacity of about 23 tons instead of its actual safe load capacity of about 3.5 tons. The designer of the NECC temporary bridge believed that the bridge's substantial vertical load design factor of safety (3.5 to 1) would also accommodate any lateral loads that the bridge would experience. As illustrated by this collapse, an increase in the vertical load capacity of a bridge may not result in a similar increase in its lateral load capacity. Therefore, the Safety Board concludes that it was inappropriate for NECC to assume lateral load capacity for the temporary bridge simply by providing a substantial vertical load factor of safety in its design calculations. Furthermore, the Safety Board concludes that had the NECC temporary bridge been designed for lateral loads such as those specified by AASHTO, the bridge would have withstood the combined debris loads that caused the collapse.

The UMD analysis of the GOP temporary bridge design concluded that this design met all applicable AASHTO group loading combinations for highway bridges. According to UMD estimates, the GOP design would have withstood a lateral load of about 39 tons before the formation of the first plastic hinge, and collapse probably would not have occurred until debris loading reached at least 65 tons. Thus, the Safety Board concludes that the GOP temporary bridge design would have withstood the combined debris loads that caused the collapse of the NECC bridge. However, because the maximum amount of debris during the flood is not known, the Safety Board could not determine whether the GOP design would have withstood all lateral debris loads during the entire flood.

Oversight of the Bridge Project

The HCE's office uses consultants in the design, construction, and review of county projects because it does not have a sufficient engineering staff for these functions. Although the HCE reviewed NECC's temporary bridge design plans for construction purposes, GOP was retained to perform a design review of the proposed alternate bridge plans. During the review, GOP raised questions about NECC's design, noting that it included no calculations for lateral loading. The HCE did not require NECC to make lateral load calculations or design modifications to address GOP's concerns. As a result, the NECC temporary bridge was constructed with little consideration for lateral loads. Had the HCE's office required NECC to perform lateral load calculations, the office would have discovered that the NECC temporary bridge design had a low lateral load capacity. The Safety Board concludes that HCE's approval of the NECC design without requiring design calculations for lateral loads resulted in the construction of a bridge that was inadequate for the lateral loading conditions imposed on it.

Also, the HCE did not submit the temporary or permanent bridge design plans to ODOT for review and approval as required by Ohio law. If the HCE had done so, the absence of lateral loading calculations might have been questioned by an engineer reviewing the plans. An ODOT bridge engineer testified during the Safety Board's public hearing that the NECC temporary bridge design would not have been approved.

Although the construction crew was not at the site on the day of the accident, the Hamilton County assistant bridge engineer stopped by to check the work status. While he was at the site, the water level increased from 13 feet to 18 feet on the gauge. He stated that he was not alarmed by this because earlier in the week (3 days before the collapse) the river gauge had read 21 feet. He testified at the Safety Board's public hearing that there were no written procedures or policies for closing bridges.

An NECC superintendent stopped by to check the construction site about 4:20 p.m. He did not notice anything unusual about the bridge but was alarmed by a telephone pole leaning precariously toward it. According to testimony at the Safety Board's public hearing, the superintendent apparently believed that he needed to get permission from the HCE's office before taking action to close the bridge. In addition, HCE employees testified that they expected to be contacted regarding a decision to close the bridge. Even though the superintendent's actions were not in response to a potential collapse of the bridge, had a procedure been in place that provided the superintendent with the authority to close the bridge, he could have closed it before the collapse.

At the time of the collapse, the HCE knew that lateral loads had not been calculated in the design of the temporary bridge and was aware of the flood conditions during the week preceding the collapse. Because the lateral design capacity of the bridge was not known, the HCE should have initiated procedures for monitoring the bridge during the flood conditions. During the initial contractual arrangements, the HCE's office and NECC should have agreed on procedures for monitoring and closing the temporary bridge structure. The Safety Board concludes that the HCE's failure to establish a policy and to develop procedures for monitoring and closing the temporary bridge during flooding conditions contributed to the severity of this accident. The Safety Board believes that the HCE should establish policies and develop procedures for bridge closure. Furthermore, the Safety Board believes that all States should require that any bridges susceptible to hydraulic and debris loading be monitored during flood events to ensure that they are closed when lateral loads exceed the design loads.

The Role of Scour

The Safety Board obtained riverbed elevations ranging from 466 feet to 480 feet from several different sources taken at different times and at various locations surrounding pile bent 2. These differences in elevations suggest that some form of scour may have occurred at or close to pile bent 2 before its collapse, at the time of its collapse, or after its collapse. However, attempts to examine the riverbed for scour where pile bent 2 was constructed were unsuccessful. The riverbed material consisted primarily of dense sand, gravel, and silt, and it was not possible from the subbottom profiles to differentiate between material that had been there before the collapse and the material that may have infilled after the flood subsided. Therefore, the Safety Board was not able to determine whether scour occurred at

pile bent 2 before it collapsed. The FHWA, NECC, and UMD analyses indicated that even allowing for the possibility of scour during the flood, differences in the unsupported length of piles had little effect on the lateral load capacity of pile bent 2. Thus, if scour had occurred, it would not have been a factor in this collapse.

Bridge Design Specifications

AASHTO.--According to the testimony from the chairman of the AASHTO Subcommittee on Bridges and Structures, AASHTO specifications for bridge design address all the forces a bridge is expected to safely withstand during the life of the structure. AASHTO does not differentiate between permanent and temporary bridges and thus, according to AASHTO, all temporary bridges should be designed in accordance with the same AASHTO loading specifications that are applicable to permanent structures. Based on the UMD analysis, the Safety Board concludes that had NECC considered AASHTO lateral loads in the design of its temporary bridge, the bridge would have withstood the debris loading that caused the collapse. Moreover, the UMD noted that the GOP temporary bridge design met AASHTO lateral group loading specifications, indicating that it is reasonable to design temporary bridges in accordance with these specifications. Therefore, the Safety Board believes that all public bridges should be designed and built in accordance with AASHTO vertical and lateral loading specifications.

Section 3.18 of AASHTO standard specifications for highway bridges states that all piers and portions of structures that are subjected to flowing water are to be designed to resist the maximum stresses induced by stream flow, floating ice, wind, and debris. The specifications provide detailed criteria for calculating the maximum expected loads and stresses for each of these conditions except debris. The specifications do not provide any guidance for calculating impact and accumulated debris loads.

According to testimony provided by the chairman of the Subcommittee on Bridges and Structures, debris loading is partly accounted for by the safety factor incorporated into the design and may also be estimated based on bridge site visits or historical data on river debris. Most structures that are built in accordance with AASHTO loading specifications for allowable working stress have a factor of safety of about 2 to compensate for unanticipated loads. However, this factor of safety is based on the assumption that the designer has tried to consider all known forces (vertical, lateral, and so forth) that may be imposed on the bridge during its service life. The factor of safety is intended to provide for variations in the different types of loads that are specifically considered.

Many permanent bridges are built with massive substructures to support the weight of the superstructure and, as a result, far exceed AASHTO criteria for lateral loads. These bridges are protected from debris loading by the inherent nature of the massive substructures. However, those bridges that do not have massive piers in the water, such as this bridge, are susceptible to being overstressed from loads caused by debris impact and accumulation. Therefore, it is imperative that all significant lateral forces, including debris loading, be considered in the design process. Because no specific guidance is provided by AASHTO for calculating debris loads, the Safety Board believes that AASHTO, in cooperation with the FHWA and the USGS, should conduct research to develop methods for estimating maximum debris loads, that is, frequency, size, and magnitude, for design purposes. The Safety Board believes that once these methods are developed, AASHTO should include in the "Standard Specifications for Highway Bridges" detailed criteria for calculating the maximum

expected debris loads and should specify analytical methods for determining the stresses imposed by impact and accumulated debris loads on highway bridges.

State of Ohio.--The ODOT has adopted AASHTO bridge specifications; however, AASHTO specifications do not differentiate between temporary and permanent bridges. Temporary structures are addressed in Section 207 of the ODOT bridge manual and in Section 502.2 of the Ohio "Construction and Materials Specifications." Section 502.2 specifies that temporary bridges "shall be designed for a loading of HS20-44 at unit stresses not to exceed 136.5 percent of those specified by pertinent sections of AASHTO 'Standard Specifications for Highway Bridges.'" According to ODOT, these specifications were promulgated so that temporary bridges are not designed to lesser AASHTO loadings, such as H-15, H-20, or HS-20. However, Section 502.2 does not similarly define lateral load requirements for temporary bridges.

The ODOT bridge engineer testified at the Safety Board's public hearing that the ODOT specifications require that temporary bridges be designed to the lateral loading specifications of AASHTO. Also, GOP calculated lateral loads when developing its temporary bridge design. The Safety Board believes that the AASHTO lateral load criteria should have been applied to the design of the NECC temporary bridge. However, the NECC did not use AASHTO lateral loading criteria for the design of the temporary bridge. The NECC and HCE stated that the AASHTO bridge specifications for lateral loads did not apply to the design of temporary bridges in Ohio. The Safety Board concludes that NECC and HCE may have misinterpreted section 502.2 of the ODOT "Construction and Materials Specifications" as not requiring the application of AASHTO lateral load specifications to temporary bridges in Ohio. To avoid misinterpretation, the Safety Board believes that ODOT should clarify the lateral load requirements for temporary bridges in its "Construction and Materials Specifications." Furthermore, the Safety Board believes that all States should require all AASHTO design loading specifications as minimum standards for all temporary, as well as permanent, bridges.

CONCLUSIONS

Findings

1. The performance of the passenger car driver before the collapse did not affect the outcome of this accident.

2. No preexisting defects in the superstructure caused the bridge to collapse.

3. The response of emergency personnel was timely and their actions were appropriate under the conditions.

4. The burn holes used for pile installation in piles 2 and 3 did not significantly reduce the lateral load capacity of the bridge.

5. The failure of the weld connecting the lower brace to the west flange on pile 1 substantially reduced the pile bent's resistance to lateral loads.

6. The collapse of pile bent 2 resulted from the formation of plastic hinges due to a combination of impact and accumulated debris loading on the upstream side of pile bent 2.

7. NECC inappropriately assumed lateral load capacity for the temporary bridge by providing a substantial vertical load factor of safety.

8. Had the NECC temporary bridge been designed for lateral loads such as those specified by AASHTO, the bridge would have withstood the combined debris loads that caused the collapse.

9. The GOP temporary bridge design would have withstood the combined debris loads that caused the collapse of the NECC bridge.

10. HCE's approval of the NECC design without requiring design calculations for lateral loads resulted in the construction of a bridge that was inadequate for the lateral loading conditions imposed on it.

11. Had a procedure been in place that provided the superintendent with the authority to close the bridge, he could have closed the bridge before the collapse.

12. The HCE did not comply with the Ohio requirement for the submission of bridge design plans to ODOT for review.

13. The HCE's failure to establish a policy and to develop procedures for monitoring and closing bridges during flooding conditions contributed to the severity of this accident.

14. Whether scour occurred at pile bent 2 before its collapse could not be determined; however, if scour did occur, it would not have been a factor in this collapse.

15. Current AASHTO specifications do not provide criteria for calculating combined debris loads in the bridge design process.

16. The NECC and HCE may have misinterpreted section 502.2 of the ODOT "Construction and Materials Specifications" as not requiring the application of AASHTO lateral load specifications to temporary bridges.

Probable Cause

The National Transportation Safety Board determines that the probable cause of the temporary bridge collapse was the selection by the Hamilton County Engineer's Office of a design by the National Engineering and Contracting Company that did not consider lateral loads and the failure of the Hamilton County Engineer's Office to promptly close the bridge when it became subject to significant debris loading. Contributing to the cause of the collapse was the failure of the Hamilton County Engineer's Office to submit the bridge design plans to the Ohio Department of Transportation for review as required by State law.

RECOMMENDATIONS

As a result of its investigation of this accident, the National Transportation Safety Board made the following recommendations:

--to the Hamilton County Engineer's Office:

> Require that bridge design capacities be determined and establish policies and procedures for monitoring and closing public highway bridges when conditions exceed the design capacity of the structure. (Class II, Priority Action) (H-90-98)

--to the Ohio Department of Transportation:

> Require that bridge design capacities be determined and establish policies and procedures to close bridge structures when conditions exceed the design capacity of the structure. (Class II, Priority Action) (H-90-99)
>
> Clarify the lateral load requirements for temporary bridges in the Ohio "Construction and Materials Specifications." (Class II, Priority Action) (H-90-100)
>
> Disseminate the facts and circumstances of this accident to each county and municipal engineer responsible for bridge design and construction in the State of Ohio. (Class II, Priority Action) (H-90-101)
>
> Develop and implement procedures to ensure enforcement and monitoring of the Ohio law (Ohio Revised Code, Section 5543) that states:
>
> > No contract for the construction or repair of a bridge, the entire cost of which construction or repair exceeds fifty thousand dollars, shall be entered into by the county unless the plans are first approved by the State Director of Transportation. (Class II, Priority Action) (H-90-102)

--to the Federal Highway Administration:

> Undertake a research program in cooperation with the American Association of State Highway and Transportation Officials and the U.S. Geological Survey to develop methods for estimating maximum debris loads for bridge design purposes. (Class II, Priority Action) (H-90-103)
>
> Establish, in cooperation with the American Association of State Highway and Transportation Officials, standard analytical methods to determine loads imposed by debris impact and by debris accumulation on bridge substructures. (Class II, Priority Action) (H-90-104)

Encourage the States to determine bridge design capacities and to establish policies and procedures to close bridges when conditions exceed the design capacity. (Class II, Priority Action) (H-90-105)

Encourage all States to require the American Association of State Highway and Transportation Officials loading specifications as minimum design criteria for all bridges open to the public. (Class II, Priority Action) (H-90-106)

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

Cooperate with the Federal Highway Administration and the U.S. Geological Survey in a research program to develop methods to estimate maximum debris loads for bridge design purposes. (Class II, Priority Action) (H-90-107)

Cooperate with the Federal Highway Administration in establishing standard analytical methods to determine loads imposed by debris impact and by debris accumulation on bridge substructures. (Class II, Priority Action) (H-90-108)

Following the development of debris loading design methodologies, include detailed criteria for calculating the maximum expected debris loads and stresses in the "Standard Specifications for Highway Bridges." (Class II, Priority Action) (H-90-109)

--to the U.S. Geological Survey:

Cooperate with the Federal Highway Administration and the American Association of State Highway and Transportation Officials in a research program to develop methods to estimate maximum debris loads for bridge design purposes. (Class II, Priority Action) (H-90-110)

BY THE NATIONAL TRANSPORTATION SAFETY BOARD

/s/ James L. Kolstad
Chairman

/s/ Susan M. Coughlin
Vice Chairman

/s/ Jim Burnett
Member

/s/ John K. Lauber
Member

/s/ Christopher A. Hart
Member

November 20, 1990

APPENDIXES

APPENDIX A

INVESTIGATION

Investigation

The National Transportation Safety Board was notified of this accident by the news media, and highway accident investigators were launched from the Safety Board headquarters in Washington, D.C. The investigation team arrived on scene about 7 p.m. on May 28, 1989.

Participating in the investigation were representatives from the Whitewater Township Fire Department, the Hamilton County Engineer's Office, National Engineering and Contracting Company, the Miami Conservancy District, the Ohio Department of Transportation, and the Federal Highway Administration.

Hearing

The Safety Board convened a public hearing in Cincinnati, Ohio, on October 3 and 4, 1989, to inquire further into the collapse of the temporary bridge. The Safety Board was concerned that State specifications governing the construction of bridges were not being adhered to and that no procedures were in place for monitoring and closing the temporary bridge during flood conditions.

APPENDIX B

ABBREVIATED INJURY SCALE*

	Drivers	Passengers	Total
Unsurvivable (AIS-6)	0	0	0
Critical (AIS-5)	0	0	0
Severe (AIS-4)	0	0	0
Serious (AIS-3)	0	0	0
Moderate(AIS-2)	0	0	0
Minor (AIS-0)	1**	1**	2
Unknown (AIS-9)	0	0	0
Total	1	1	2

* Abbreviated Injury Scale refers to the abbreviated injury scale (1985) of the American Association for the Advancement of Automotive Medicine.

** Drowning is not considered an injury under the Abbreviated Injury Scale.

* U.S. G.P.O.:1991-281-626:20030