July 1993 Flood Damage to US-71 Bridge Over Brushy Creek, Carroll County, Iowa: Case Study

W. CURTIS MONK

The flood of 1993 inundated much of the upper Mississippi River Basin, resulting in significant damage at numerous sites in Iowa. A record flood on Brushy Creek resulted in failure of the US-71 bridge 14.5 km south of Carroll, Iowa. Development of a computer flow model at the site and analysis of scour for the failed bridge and its replacement are included. On the basis of data from plans, topographic maps, and a United States Geological Survey (USGS) crest stage gage at the site, the computer model was developed using WSPRO (a USGS/FHWA computer program for Water Surface Profile Computations). To construct the computer model, cross sections were developed, and Manning's roughness coefficients *n* were assigned, such that known stage elevations were produced for a given flow. The model was then used to estimate flood flows during the event. Scour was estimated using the WSPRO data, soil boring data, and procedures from HEC-18, *Evaluating Scour at Bridges.* Long-term aggradation or degradation, contraction scour, and local pier scour were all evaluated. The case study allowed verification of HEC-18 procedures and formulas based on an actual flood event versus a laboratory flume study. The scour analysis predicted scour below the pile tips at the piers, and the bridge failed when the piers were undermined—good evidence of the validity of HEC-18 scour calculations. The computer program WSPRO and the scour procedures from HEC-18 as analysis tools that can be used to assess stream flows, scour potential, and design adequacy are presented.

The flood of 1993 inundated much of the upper Mississippi River Basin. According to the United States Geological Survey (USGS) Circular 1120-B, "The magnitude of the damages in terms of property, disrupted business, and personal trauma was unmatched by any other flood disaster in United States history. Property damage alone is expected to exceed $10 billion" (*1*). Flooding in Iowa was similar to that of the rest of the upper Mississippi River basin, in the magnitude of damages and magnitude of the floods themselves. It was reported, "Only the floods of 1851, five years after Iowa statehood, appear to be possibly of similar magnitude to the floods of 1993" (*2*).

There was damage to roads and bridges in all 99 counties. Damage to federal aid highways and bridges occurred at 564 sites in 85 counties resulting in an estimated repair cost of over $21.4 million (Monk, W. C. Field Disaster Report. FHWA, Iowa Division, unpublished, 1993).

These record floods provide engineers the opportunity to review the performance of designs under the most severe conditions—conditions often designed for, but which seldom occur during the life of the structure. The floods tested many highways and bridges to the maximum; many sustained damage, but few failed.

FHWA, Iowa Division, P.O. Box 627, Ames, Iowa 50010.

This report was adapted from a report that reviewed the performance of federal aid facilities at three sites in central Iowa as part of a master of science degree by the author. The case study of the flooding and damages at the site provides insight into design parameters that must be considered in future designs. The study uses analysis tools that are readily available and can be used to evaluate stream flow and scour potential at other sites.

CASE STUDY

Location

The site of the case study is located at the US-71 bridge over Brushy Creek, 14.5 km south of Carroll, Iowa, in Carroll County (Figure 1).

Existing Conditions

The existing structure was a 27.4 m × 8.5 m three-span, prestressed, concrete beam bridge. It was constructed in 1957 to replace a 27.4 m × 6.1 m pony truss built in 1925. The abutments from the pony truss were widened and used in the replacement bridge. The abutments were high abutments supported on timber piles. The piers were pile bent type consisting of 5 to 406 mm^2 prestressed concrete piles with a concrete cap to support the superstructure. Plans called for 12.2-m piles driven to full penetration, if practicable, but to at least 285 kN bearing. Full penetration would have placed the pile tip at 374.7 m above mean sea level as calculated from the original contract plans. That would have provided about 7 m penetration below streambed, with about 2.5 m penetration into a gravel layer. Individual pile tip data are not available; however, because of the pile type and the dense gravel layer near the design tip, it is reasonable to expect that the piling achieved refusal at or near the design tip elevation. A situation plan for the site is shown in Figure 2.

This study site is located just west of the west edge of the Wisconsin glaciation into Iowa (*3*). The stratigraphy of the site is shown by soil borings taken near the roadway centerline and adjacent to the stream. The borings are shown in Figure 3. The borings indicate a layer of stiff silty clay, which is most likely colluvial loess washed in from the uplands. The sand layer that trends to gravel at 377 m elevation is interpreted as glacial outwash from the Des Moines lobe of the Wisconsin glacier. The glacial clay shown is probably pre-Illinoin (*3*).

Brushy Creek is a meandering stream founded in alluvium; it drains about 117 km^2 and has a stream slope of 1.6 m/km above the

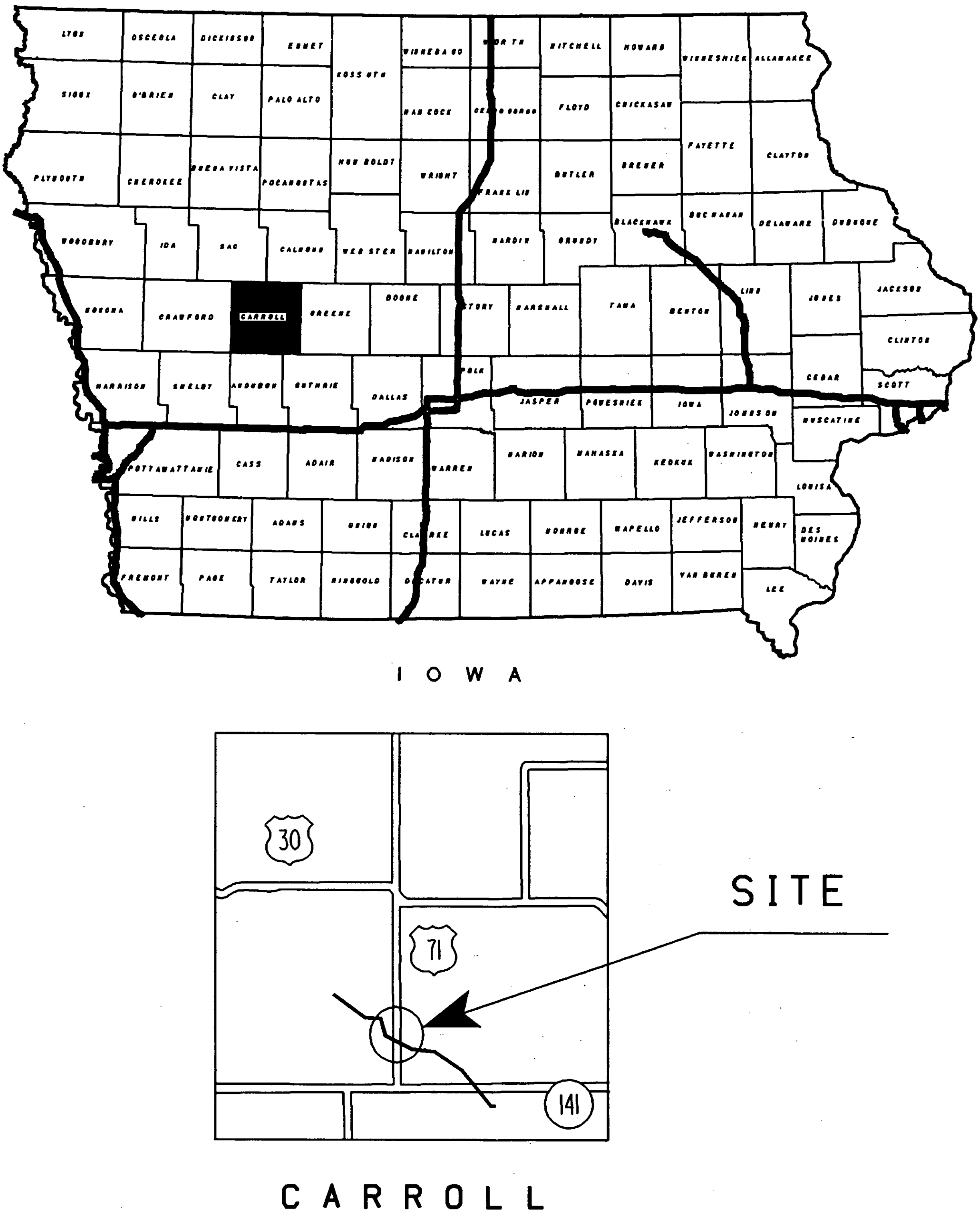


FIGURE 1 Location map.

bridge site. Figure 4 provides some topographic details for the study site. The topography is rolling hills. The floodplain of the stream is 274 m wide at this location, which is excessive for the size of the modern drainage area. This evidence, with the presence of coarse gravel and boulders in the borings, supports the interpretation that this stream served as a glacial melt-water outlet. Furthermore, such a melt-water deposit would be expected to be fairly extensive and not an isolated pocket.

The bridge was built so that the stream channel was parallel to the piers and abutments. However, over the years channel migration resulted in channel flow striking the piers and the south abutment at approximately 5 degrees (Figure 2).

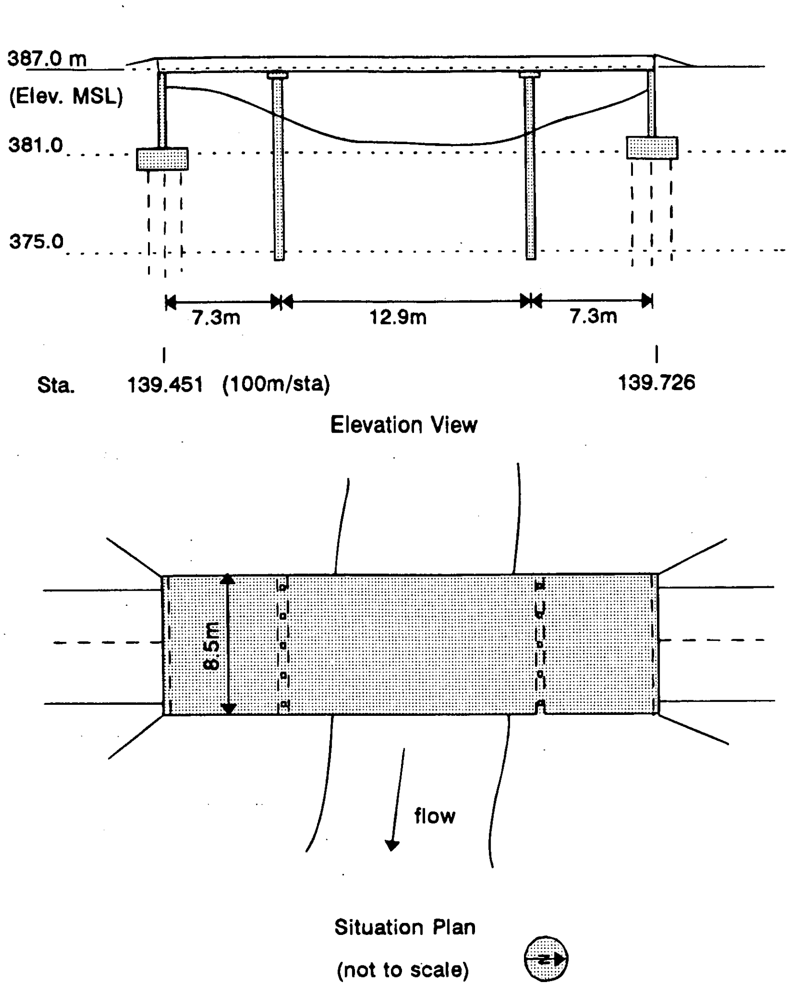


FIGURE 2 Situation plan for existing bridge.

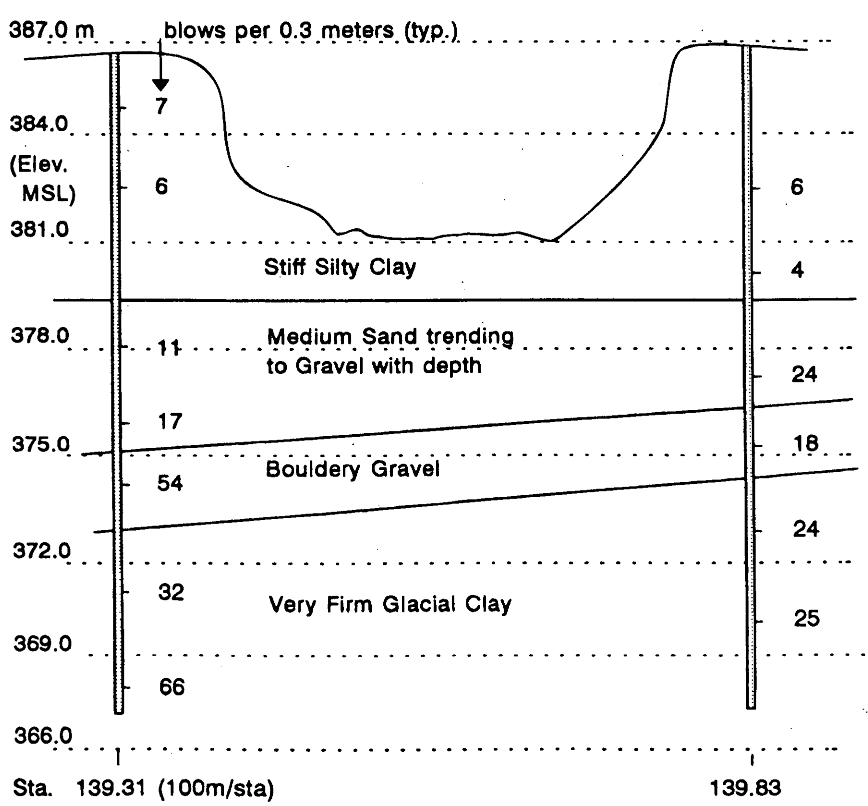


FIGURE 3 Soil borings for US-71 Bridge (Iowa DOT, 8/1993).

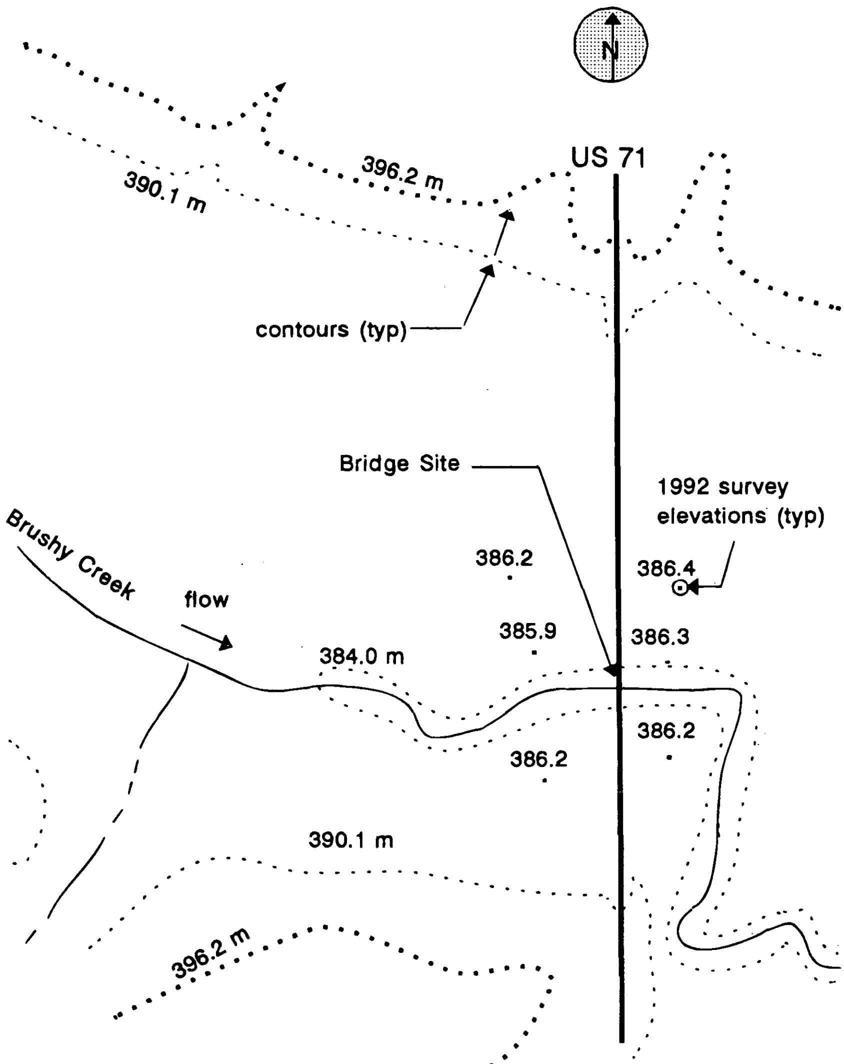


FIGURE 4 Topographic details of site.

Flood Data

A crest-stage gage that records gage heights at peak flows was installed by USGS on the downstream side of the bridge. The annual-peak list for this gage is shown on Table 1 (personal communication from USGS). The gage heights are based on an arbitrary datum that is tied to an Iowa State Highway Commission bench mark at the old bridge. In addition to crest-gage records, Table 1 shows gage heights for high water marks of the 1993 flood upstream and downstream at the site. USGS recorded these high water marks after the 1993 flood destroyed the crest-stage gage. At the time of this study USGS had not calculated flows at this site except for the 2 years noted in Table 1. At a later date they did calculate the discharge for the 1993 flood and published the data in the Water Resource Data for Iowa, Water Year 1993 (*4*).

Flow Model

To evaluate the 1993 flood at this site it was necessary to estimate the flow. By knowing the flow, the flood could be compared with previous events, scour calculations could be performed, and causes of the failure could be assessed. No flow measurements were taken at the site during the flood, so indirect methods were used to estimate the flow.

The USGS data from Table 1 and a computer program that estimates water surface profiles (WSPRO) were used to develop a computer model that would simulate the flows at this site (*5*). To construct a computer model, cross sections were developed, and Manning's roughness coefficients *n* were assigned, such that known stage elevations were produced for a given flow. The USGS data in Table 1 provided flows and stage elevations for two events, which were used to calibrate the model. Because the existing bridge was scheduled for replacement before the flood, cross sections had been taken in 1992 in the area of the bridge and were available during this study from the proposed bridge plans. Cross sections and stream slope data were obtained from these proposed bridge plans. USGS topographic maps were used to check stream slope and determine floodplain width and elevations near the floodplain boundary. Manning's roughness coefficient was estimated based on discussions with the Iowa DOT and USGS and on guidelines from the USGS and FHWA (*5,6*).

TABLE 1 Annual Peak Flows at Brushy Creek (USGS)

STATION 05483318 BRUSHY FORK CREEK NEAR TEMPLETON, IOWA

AGENCY:	USGS	STATION LOCATOR		DRAINAGE AREA:	117 km^2
STATE:	19	LAT.	LONG.	CONTRIBUTING	
COUNTY:	027			DRAINAGE AREA:	km^2
DISTRICT:	19	415645	0945245	GAGE DATUM:	359.584 m
				BASE DISCHARGE:	

Water Year	Date	Peak Discharge (m^3/sec)	Discharge Codes	Gage[a] Height (m) (Typical)	Gage Ht Codes
1966	06/12/66			27.01	
1967	06/07/67			27.04	
1968				--	no peak
1969	07/26/69			26.51	
1970	05/13/70			25.58	
1971	03/13/71			26.76	1--gage height affected by backwater
1972	10/30/71			26.36	
1973	09/26/73			27.01	
1974	06/23/74	150.9		27.72	
1975	04/27/75			26.13	
1978	09/12/78	52.4		26.69	
1979	03/19/79			26.39	
1982	05/26/82			26.09	
1984	06/13/84			25.57	
1986	06/30/86			27.01	
1988	06/07/88			26.77	
1989	07/08/89			23.91	
1990	06/16/90			27.61	
1991				--	no peak
1992				--	no peak
1993	07/10/93			28.52	= ave hwm d/s
				29.78	= ave hwm u/s

[a]Add to Gage Datum to get elevations in report.

It is desirable to have site-specific surveys for a substantial reach of the stream being analyzed to achieve the most accurate results. However, one intent of this study was to analyze the site using readily available data. Therefore, the flood at this site was analyzed using the available information, and further surveys were not obtained.

Modeling the site was difficult because of the 90-degree bend in the main channel 60 m downstream from the bridge and the short reach being modeled. The use of conventional values of n for a meandering channel would not produce the stage elevations that USGS had recorded in 1974 and 1978. The final model included a sharp increase in the roughness coefficient in the main channel just upstream from the bend. The impact of sharp bends on n is discussed in guidelines for estimating values of n (*6*). The use of a large value for n at the downstream cross section is appropriate because the model represents only a short reach of the stream, that is, the 60 m on either side of the bridge. Therefore, it was necessary to culminate all of the downstream effects, including the 90-degree bend, at one cross section, resulting in a high value of n at the downstream cross section. Conventional values of n were used elsewhere in the cross sections to model the effects of the flow depths, low conveyance zones, and varying roughness. These n values were adjusted to fine tune the model.

The 1974 and the 1978 flow events were diverse enough to allow calibration of the model. The 1978 event was confined to the main channel and allowed development of a model that incorporated the effects of the bend. The 1974 event, which included 0.9 m of overbank flow, was then used to develop the remainder of the model, including overbank flow conditions.

The actual stage data and calculated flows are compared with predicted values in the following table:

Year	Flow (m^3/sec)	Stage (m) Actual	Model
1974	150.90	387.30	387.30
1978	152.40	386.27	386.30

Comparison of 1993 Flow with Past Events

After calibration, the computer model was used to estimate the peak flow, velocity, and flow distribution in the cross section for the 1993 event. The results were used to compare the 1993 event with past events and to estimate scour.

USGS regression formulas developed for Iowa were used to estimate the flows for recurrence intervals of 50 and 100 years at the site (*7*). Based on the drainage area and the region of the state, the regression formulas were used to estimate a flow of 154 cm for a Q_{50} flood and 181 cm for a Q_{100} flood at the site. The peak flow estimated with the model for the 1993 flood was 552.2 m^3/sec or 3 times that of the Q_{100} flood predicted by using the regression formulas. The results of USGS calculations of the discharge at this site for the July 1993 flood confirm the peak flow estimated with the model. Their independent calculations published after completion of this study indicate a peak flow of 538 m^3/sec at this site (*4*).

It was suggested (*8*) that a value of 1.7 times the Q_{100} flood be used as an estimate of Q_{500}. Therefore, it is likely that the recurrence interval of this flood was in excess of 500 years. The highest previous recorded flow at this site was 150.9 cm, occurring in 1974 with a recurrence interval of about 50 years. Another approximately equivalent flow occurred in 1990 (Table 1).

Scour

Scour at a bridge site can include long-term aggradation or degradation, contraction scour, and local scour at piers and abutments. These processes are complex and often interrelated. When a structure is analyzed, whether existing or a new design, for scour, these processes must be evaluated for conditions as they exist now and how they may change in the future because of changes in the river environment.

The long-term aggradation or degradation of this site was assessed by reviewing biennial reports from inspections that the Iowa DOT conducts as part of their bridge safety program. Cross sections taken during these inspections indicate that the stream channel has remained fairly constant since the bridge was constructed. Cross sections taken in 1992 to design a replacement structure indicate the streambed elevation is within 0.15 m of what it was in 1957.

Contraction scour and local scour at the piers were analyzed using recommended procedures discussed in *Evaluating Scour at Bridges* (HEC-18) (*8*). Local scour at the abutment was not analyzed because it did not appear to have contributed to the collapse. The HEC-18 procedures (*8*) recommend calculating contraction scour and adding local scour as though the two are independent unless contraction scour is more than 1.5 to 1.8 m. In that case, the stream cross section is adjusted for contraction scour before calculating local scour. The second method accounts for the reduced velocity through the bridge opening once contraction scour occurs. The second method was used at this site because the wide floodplain could result in significant contraction scour.

The alignment of the flow with the bridge can significantly affect scour at the structure. The main channel is almost parallel (5-degree angle of impact) to the axis of the substructure units. However, based on the USGS topographic map it is apparent that during a flood of this magnitude the angle of impact is much more severe.

The computer program WSPRO (*5*) allows the user to divide the floodplain into sections and thereby estimate the distribution of flow in the main channel and the floodplain. A flood stage of 389.4 m, which from Table 1 is the high water elevation recorded by the USGS upstream of the bridge, results in over 3.0 m of flow in the overbank area of the valley cross section. Because the roadway grade constricts the floodplain, all of the overbank flow is forced through the bridge opening until roadway overtopping occurs. Using WSPRO it was determined that overbank flow was carrying over 75 percent of the total flow, and 76 percent of that overbank flow was carried on the north floodplain. With this magnitude of flow in the north floodplain, it is reasonable to estimate the angle of impact as the angle between the axis of the overall valley section and the pier axis. This results in an impact angle of 20 degrees.

The equations for scour along with the discussion of terms are long and are not included here. However, a complete discussion of terminology and methods of calculating contraction and local pier scour can be found elsewhere (*8*).

Scour calculations were performed for the 1993 and the 1974 floods. The results, along with other pertinent information, are included in Table 2.

It should be noted that local pier scour is calculated assuming that contraction scour has already occurred. It would occur simultaneously with contraction scour and would most likely be more intense because of the downward direction of the flow vortex around the pier (*8*). However, this interaction would not affect the ultimate scour.

Scour depths as calculated by HEC-18 procedures (*8*) are ultimate scour depth. That is, given enough time during one event or

TABLE 2 Results of Scour Calculations

Year	Contraction[a] Scour Depth (m)	Local Pier Scour (m)	Elevation Scour (m)	Elevation Pile Tip (m)	Remaining Capacity (kN)[b]
1974	2.9	1.8	379.0	374.7	444.8
1993	6.4	2.7	374.6	374.7	0

[a]Contraction scour depth is measured from average ground elevation in the bridge opening prior to scour which was 383.7 m.
[b]Estimated for design pile lengths using Iowa DOT procedures.

several smaller floods, the calculated scour depths will occur. The calculated scour depths for the 1993 flood indicate scour below the pier pile tips, and the bridge failed, which is good evidence that the calculated scour occurred. This indicates that the 1993 event was of sufficient duration to attain its ultimate scour depth in one event.

Scour calculations for the 1974 event indicate scour to 2.7 m below the streambed. It is difficult to judge whether this amount of scour occurred. It could have scoured and then silted in during the falling stage of the flood, or the duration of the flood may not have been sufficient to attain the calculated depth of scour. Some scour undoubtedly occurred during both the 1974 and the 1990 events. Once bed material scours and is silted back in during the falling stage, it will scour more rapidly the next time. This most likely contributed to the scour depths attained during the 1993 event.

Failure

Failure at this bridge was caused by scour at the piers. Several factors contributed to the failure: the magnitude of the flood, the angle of attack of the flow during the large flood, contraction scour, and the pile tip elevation.

The largest contributor to failure was the magnitude of the flood. Therefore, although one can learn from this failure, one must be careful about our conclusions. Under current standards, the design would be checked to ensure a factor of safety of one for a superflood. A superflood is defined as a flood of 1.7 times a Q_{100} flood (*8*). As estimated with the model and confirmed by independent USGS calculations (*4*), the 1993 flood was 3 times a Q_{100} flood. In most instances it would not be cost effective to design for a flood of that magnitude.

The angle of attack was a significant contributor to scour depth. A 20-degree angle of attack was estimated for this structure when it was subjected to large overbank flows. This resulted in local pier scour 1.8 times greater than if piers were parallel to flow. In addition the depth of flow exceeded the low superstructure elevation, resulting in a pressure flow situation, which also increases scour (*8*). Although local pier scour played an essential role in the failure, it did not cause the majority of the scour: contraction scour did (Table 2).

Contraction scour occurred because a floodplain that was 274 m wide was constricted to a 26.5-m bridge opening. As previously indicated, 75 percent of the 1993 flood was being carried in the overbank area and was forced through the restriction at the bridge site. The consequence of such a restriction is apparent: as the flow was forced through the constriction, velocities increased from 0.7 m/sec upstream to 3.5 m/sec through the bridge opening. This resulted in 6.4 m of scour (Table 2). Calculations using HEC-18 procedures indicate that scour would have been greater except for natural armoring (increase in particle size) in the form of a gravel strata left by an ancient glacier (*4,8*).

It is interesting to note that without the combined effects of both types of scour the bridge may not have failed. Using HEC-18 procedures (*8*) contraction and local scour can be considered independently. At maximum flow and velocity, but without contraction scour, local pier scour would have reached an equilibrium depth at elevation 378.9, leaving 4.2 m of the piles still embedded. Contraction scour ceased once it hit the gravel strata at elevation 377.3, leaving 2.6 m of the piles embedded.

The bridge would certainly have been stable with 4.2 m of pile embeddment and possibly with 2.6 m., if there was no debris pileup, because most of the pile bearing was end bearing as calculated using Iowa DOT procedures (Dirks and Cam, unpublished data). However, the scour analysis indicates that after contraction scour from the 1993 flood created a large hole upstream and through the bridge, local scour at the piers continued to remove material even at reduced velocities. This was the result of the depth of flow (10.8 m), which has a major influence on the depth of local scour. The combination of both processes resulted in the bridge failure.

The tip elevation of the piles did not provide sufficient embedment for a flood such as the 1993 event. However, it was adequate for two separate Q_{50} events (1974 and 1990). The bridge was most likely designed based on a Q_{50} flow. It is unknown whether the bridge was analyzed for scour during design. Scour was probably considered, if not strictly analyzed. The impact of scour at bridges has been considered in design for many years. Because of recent failures due to scour and guidelines from FHWA, engineers have begun to analyze more rigorously the impacts of scour. Pile tip elevations should not be determined solely on bearing capacity requirements and the present streambed configuration.

Replacement Design

The replacement design is a 61.3 × 13.4 m prestressed concrete beam bridge with two encased pile bent piers. The length of the bridge is partly due to instability of the loess layer that requires 3:1 abutment slopes. According to Iowa DOT an estimated maximum scour to elevation 378 m will be used for foundation design. Piers will be a pile bent type with steel H piles driven to bearing and encased in a concrete wall above the streambed. Abutments will be founded on steel H piles driven to bearing. Riprap will be placed in the main channel 45.7 m upstream and downstream of the bridge. The roadway grade will be raised across the entire flood plain and will have a low elevation of 389.5 m. A situation plan for the replacement bridge is shown in Figure 5.

The replacement design was analyzed for a flow of 181 cm (Q_{100}). It was checked to ensure a factor of safety of 1.0 for a superflood (Q_{500}), which was taken as 340 cm or about 1.9 times Q_{100}. The results are shown in Table 3.

As Table 3 indicates, the piers would have adequate capacity. However, according to Iowa DOT guidelines, excessive column lengths would be exposed. In addition, scour of this magnitude may result in slope failure due to a steepened abutment slope overlying the layer of weak colluvial loess, reflected as silty clay with SPT = 4 in Figure 3. Therefore, scour will need to be limited by placement of riprap not only in the channel but also up the abutment slope to protect the piers from local scour. The riprap will be sized to armor the streambed and bank, preventing initiation of either contraction or local scour. The riprap should be carried around the embankment slope on the north floodplain to prevent redevelopment of the scour hole that occurred there (Figure 6). Numerous model studies have shown that this location is subject to the most severe contraction scour (*9*).

CONCLUSION

The intent of this paper was to analyze the effects of the 1993 flood on the US-71 Bridge south of Carroll, Iowa. In a case study of the site, the extent of the scour was analyzed, causes of the scour were discussed, and important future design considerations were highlighted. The use of the computer program WSPRO and scour procedures from HEC-18 were presented as analysis tools that can be used to assess stream flows, scour potential, and design adequacy.

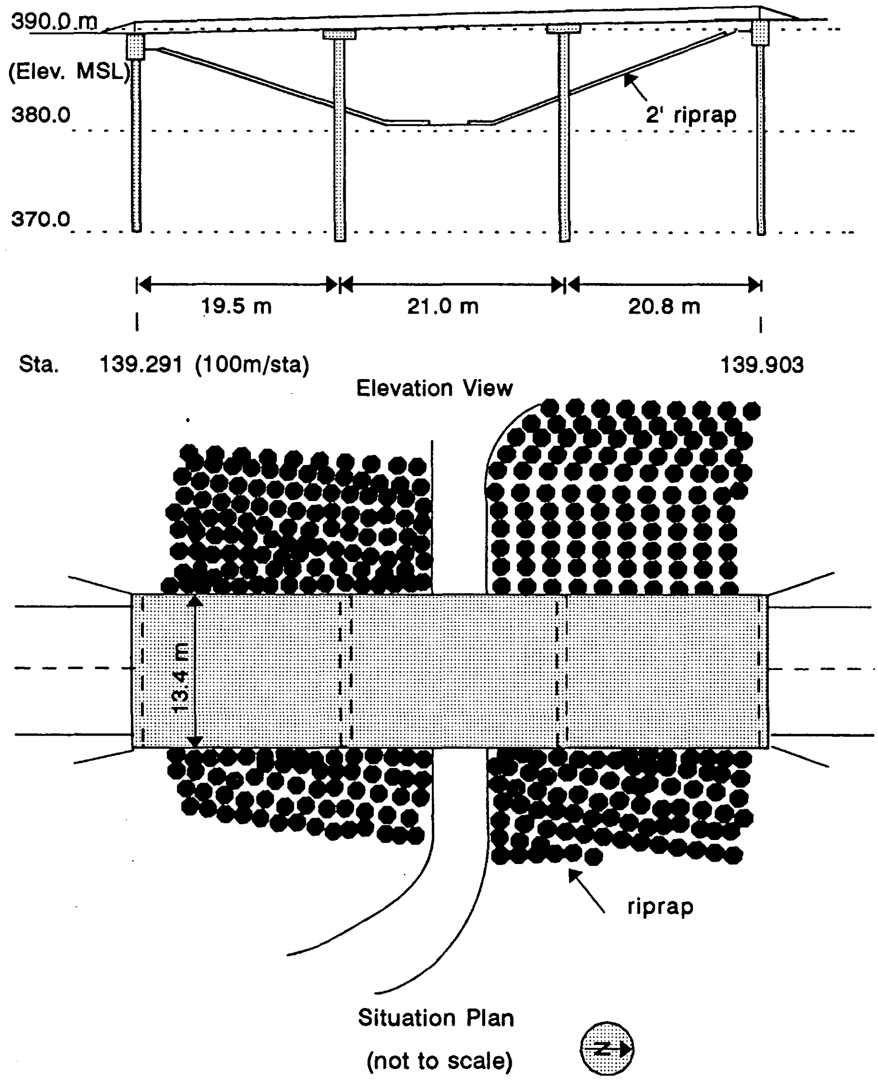


FIGURE 5 Situation plan for new bridge.

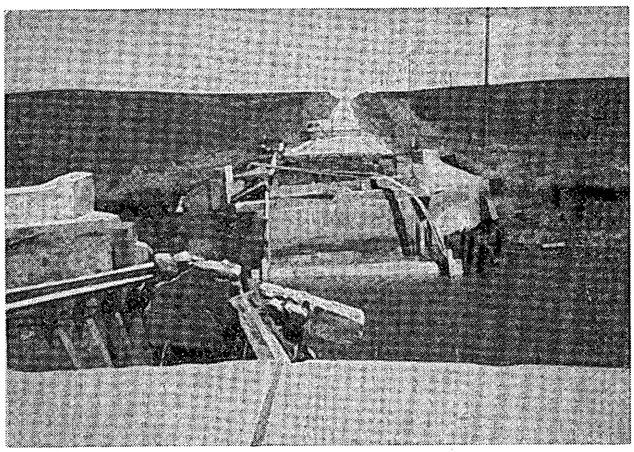
TABLE 3 Results of Scour Calculations for Replacement Bridge

Q.	Flow (m^3/sec)	Contractionscour[a] (m)	Local scour (m)	Elevation Scour (m)	Elevation Pile Tip (m)	Bearing Cap.[b] (kN)
Q100	181	0	4.4	379.7	367	355.8
Q500	340	0.7	6.4	376.9	367	329.1

[a]Contraction scour measured from elevation 384.0 m.
[b]Estimated using Iowa DOT procedures .

Because of the magnitude of the 1993 flood, one must be careful of conclusions. In most instances it would not be cost effective to design for an event such as the 1993 flood, which was estimated to be more than 3 times a Q_{100} flood at this site. However, the study site provided insight into elements of the design that should be considered even at smaller flows.

The importance of considering such elements as the angle of flow striking the structure during flood events, constriction of wide floodplains, changes in the river environment, and critical design details such as pile penetration must not be underestimated. Each of these elements in the design must be evaluated based on present conditions as well as possible future changed conditions.



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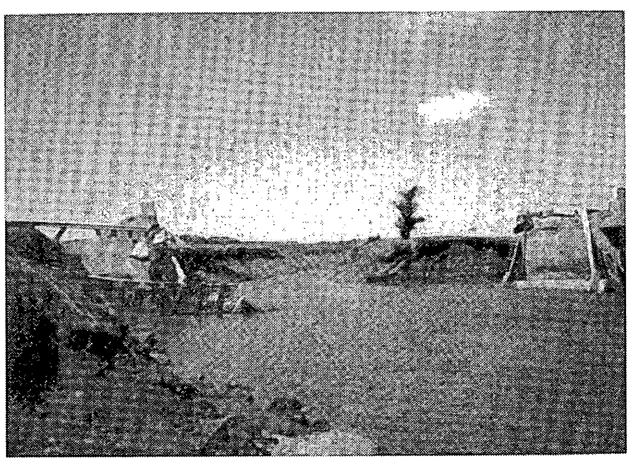

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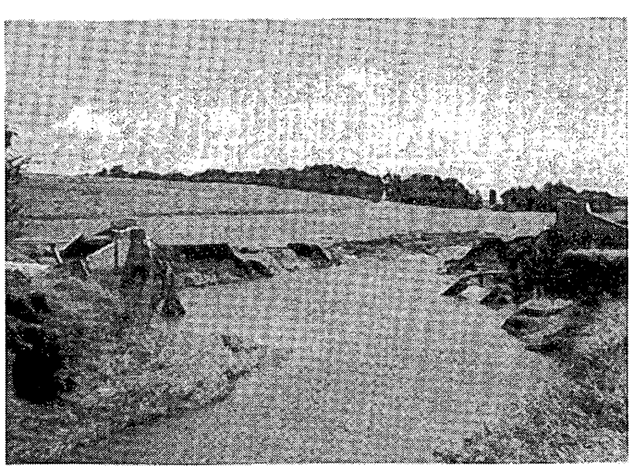

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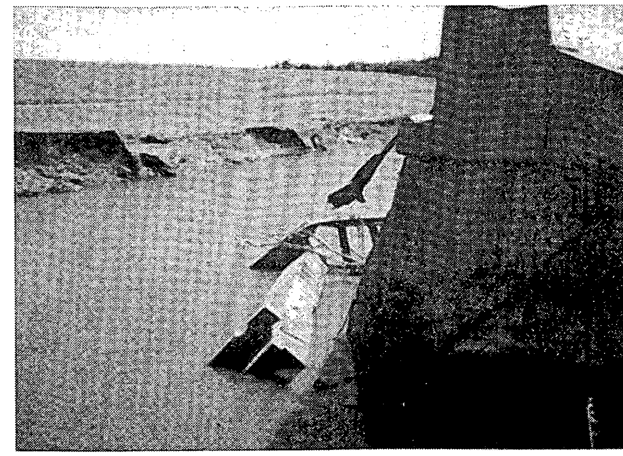

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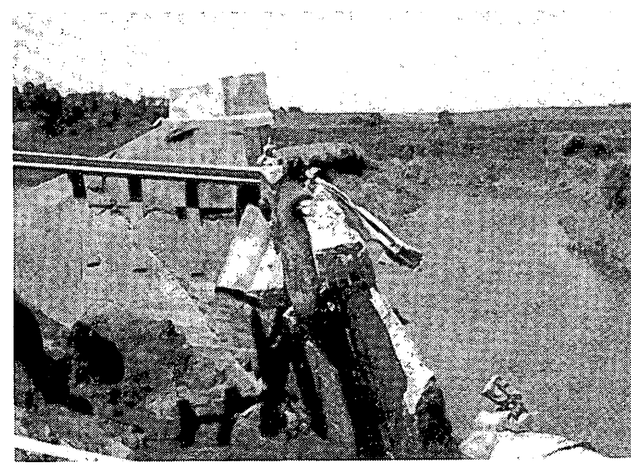

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FIGURE 6 Site after flooding: (*a*) looking north along centerline of highway, (*b*) looking north (note scour hole upper right), (*c*) looking downstream (note blowout of channel), (*d*) south abutment (note piers failed and beams fell from abutment seat), and (*e*) looking upstream (note broad flat floodplain oversized for modern stream).

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