ANALYSIS OF STREAMFLOW IN THE MAGBY CREEK BASIN NEAR COLUMBUS, MISSISSIPPI

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INTRODUCTION

Part of the Magby Creek basin east of Columbus, Lowndes County, Mississippi, was inundated by floodwaters in February and December 1990. The floods were nearly identical in both discharges and water-surface elevations. The recurrence interval for the two 1990 floods was about 3 years. Lehmberg Road (Figure 1) was overtopped by floodwaters during both of the 1990 floods, and local home owners and business people were concerned about the frequency and severity of the flooding. The U.S. Geological Survey (USGS), in cooperation with the Mississippi Department of Transportation (MDOT), performed a study of the surface-water flow patterns within the Magby Creek basin in an effort to simulate the flows of the 1990 floods and to analyze possible alternatives to decrease the severity of future floods in the basin.

Purpose and Scope

This paper describes the results of a study in which a two-dimensional finite-element surface-water model was used to: 1) simulate the two floods of 1990 and the 50- and 100-year design floods for existing conditions; and 2) simulate the effects of a hypothetical floodflow diversion levee upon flows within the Magby Creek basin upstream from Lehmberg Road. Also included in the discussion are the general theory, data requirements, and techniques used to calibrate and verify the model selected for the flow simulations.

Acknowledgments

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DESCRIPTION OF STUDY AREA

The study area is a 2.3-mile reach of the Magby Creek basin located between State Highway 50 and U.S. Highway 82 east of Columbus, in Lowndes County, Mississippi (Figure 1). The basin ranges in width from 0.3 to 0.6 mile. Magby Creek flows westward into Luxapallila Creek about 1.2 miles downstream of the study area. The average slope of the basin in the study area is about 11 feet per mile. Near the upstream (eastern) end of the study area, a distributary to Magby Creek was constructed to divert floodflows from the Magby Creek basin into the Vernon Branch basin in an attempt to lessen the magnitude of floods occurring at Lehmberg Road. Local residents state that the distributary was dug about 70 years ago. The distributary diverts flows from the Magby Creek channel south to U.S. Highway 82 and then west parallel to the north highway embankment (Figure 1).

A shallow flood plain begins for the distributary where it leaves Magby Creek. The distributary flood plain is contained within the Magby Creek flood plain, but flows do not intermingle until about a 5-year flood occurs. A 5-year flood has a 20 percent chance of being equaled or exceeded in any given year.

Old Mill Road crosses the flood plain from north to south near the middle of the study area. There are two bridge openings in Old Mill Road (Figure 1). One bridge is for Magby Creek near the northern edge of the flood plain, and the other is for the Magby Creek distributary near the south edge of the flood plain. According to local residents, Old Mill Road was overtopped by floodwaters during the floods of March 1970 and April 1979.

Lehmberg Road crosses the Magby Creek flood plain about 1,600 feet downstream of the downstream end of the study area (Figure 1). Lehmberg Road was overtopped by floodwaters during the 1970 and 1979 floods and was also overtopped during the floods of February and December 1990.

A private drive connects Old Mill Road and Lehmberg Road and is located in the Magby Creek flood plain in the west one-half of the study area (Figure 1). The drive is not elevated above the natural flood plain and has no significant effect upon floodflows.

Six ponds are located in the study area along the north embankment of U.S. Highway 82 (Figure 1). The ponds range in size from about 7 to 15 acres and the heights of their levees range from about 1 to 2 feet. Three of the ponds were built west of Old Mill Road near the 200-ft Magby Creek distributary bridge opening in U.S. Highway 82. The ponds are referred to from west to east as pond 1 to pond 6 (Figure 1).

The combined drainage area of Magby Creek and the Magby Creek distributary at Old Mill Road is about 42.1 square miles. The length of the basin upstream of Old Mill Road is about 18 mi and the average slope between points 10 and 85 percent of the length is about 8.4 feet per mile. Based on the regressions presented by Landers and Wilson (1991), the 50- and 100-year discharges are about 8,030 and 9,130 cubic feet per second, respectively.

DESCRIPTION OF SELECTED MODEL

All floodflows are three-dimensional in that the flow direction at any given point within the system is a resultant of its three component velocity vectors in the X, Y, and Z directions. Three-dimensional analyses of floodflows are complex and time consuming and may result in excessive project costs. However, most floodflows are dominated by velocities in the horizontal plane and can be acceptably simulated in one or two dimensions.

The complexity of the floodflow hydraulics varies with the physical and hydrologic characteristics of the study area. For example, if a study area were a straight reach of flood plain with no constrictions, bridges, or obstructions, the flow velocity vectors would principally be in one direction (dimension). Floodflows could then be simulated using a one-dimensional step-backwater model. Flows become more difficult to simulate as they pass around obstructions and through bridges, and, in many cases, accurate simulations of such flows cannot be obtained with a one-dimensional model.

Because of the multiple bridge openings and shallow flood-plain flows around pond levees, the existing flow patterns within the study area could not be adequately simulated with a one-dimensional model. The Finite Element Surface-Water Modeling System for 2-Dimensional Flow in the Horizontal Plane (FESWMS-2DH) (Froehlich 1989) was selected as an appropriate model for simulating the two-dimensional flows within the study area.

General Theory of the Model

FESWMS-2DH is a vertically (or depth) averaged model used to simulate two-dimensional flows in the horizontal plane. FESWMS may be run for either time dependent or steady state conditions. It consists of the three following programs:

1) DINMOD (pre-processing program for data input, plotting, and checking),
2) FLOMOD (flow modeling program), and
3) ANOMOD (post-processing program for analyzing and plotting FLOMOD output).

When using a finite-element model, the study area is divided into triangular and quadrangular elements that form a grid (Figure 2). Nodes are located at the corners and mid-sides of the elements and are assigned coordinates and elevations. The model uses the Galerkin finite-element method to solve three partial-differential equations representing conservation of mass and momentum for each grid element (Lee and Froehlich 1989). The finite-element grid used in this study has 1,780 elements and 8,047 nodes (Figure 2).

Because the system of flow equations is nonlinear, a Newton iterative method of approaching a solution is used (Gilbert and Froehlich 1987). The theory of the model is beyond the scope of this paper; however, a detailed explanation of the theory is provided in the research report by Lee and Froehlich (1989).

Data Requirements of the Model

A finite-element network should be carefully designed so that mass is conserved within the system. The finite-element grid needs to be more refined in areas where changes in velocity or bathymetry are substantial than in areas where changes are gradual.

Topographic Data

Topography is a major factor in the design of the finite-element grid. In this study, six flood-plain cross sections (Figure 2) were surveyed by the Starkville project office of the MDOT during the winter of 1991-92 and the resultant data were utilized for the grid development. The MDOT also provided survey data locating the pond levees in relation to the U.S.

Highway 82 stationing. Topographic data from USGS topographic maps were used to supplement the MDOT survey data. Roadway embankment elevations (for weir flow computations) and bridge openings were surveyed by USGS personnel in March 1992 to complete the topographic data requirements of the model.

Hydraulic Data

Hydraulic data used by the model are discharges, flood elevations, and roughness coefficients. Discharge data are discharges which correspond to the floods being simulated. Because of the large magnitudes of the floods simulated in this study, sustained peak discharges are probable. Therefore, time-dependent effects were assumed to be negligible and steady flow conditions at the peak discharge were simulated.

Flood elevations corresponding to the flows being modeled can be determined from field surveys, surface-water gaging stations, or visual observations. The elevation data are used by the model as boundary conditions or they can be used to verify model calibration.

In the FESWMS finite-element grid, each element has a roughness value assigned for flow calculations. Roughness coefficients (Manning's "n") were selected in the field in October 1991 and were estimated to represent winter and early spring conditions when vegetation is sparse and flooding is most likely to occur.

Boundary Conditions

Boundary conditions are established around the perimeter of a finite-element network and are represented by either closed or open boundaries. Boundary conditions are usually determined from measurements, observations, or surveys of hydraulic data.

Closed boundaries do not allow flows to pass through and are used to represent shorelines, obstructions, levees, and embankments. Boundaries representing shorelines should be located near the edges of the areas of flow so that the computed flow depths are always positive. Flow depths that change from positive to negative during computations produce erroneous results. The locations of the closed boundaries representing the shorelines in this study were estimated using water-surface profiles generated using the model WSPRO [a one-dimensional step-backwater model used for computing water-surface profiles (Shearman 1990)]. For the simulations in this study, all solid boundaries are set up for tangential slip conditions. The tangential slip conditions force all flows adjacent to the solid boundaries to flow parallel to the boundaries. Flows can also be allowed to pass over solid boundaries to simulate weir flows over embankments.

Open boundaries are established where flows enter or leave the finite-element system. In this study, open boundaries are located at the upstream and downstream ends of the study area and the section south of the U.S. Highway 82 bridge where the Magby Creek distributary flows leave the study area to approach Vernon Branch. The boundary conditions at the upstream end of the study area are the discharges for the different flows being simulated. The boundary conditions at the downstream end of the study area are normal water-surface elevations estimated from slope-conveyance computations. Step-backwater analyses (in the upstream direction) performed with WSPRO indicated that water-surface profiles for flood discharges computed with different downstream starting elevations converged about 2,500 feet upstream of the downstream end of the study area. The U.S. Highway 82 bridge is about 5,000 feet upstream of the downstream end of the study area. Therefore, any error in the boundary conditions at the downstream end of the study area does not affect the flows near the U.S. Highway 82 bridge. The downstream boundary conditions computed from the slope-conveyance computations are 189.8, 191.6, and 191.9 feet for the floods of 1990 and the 50- and 100-year floods, respectively. The changes in discharges approaching Lehmberg Road caused by the simulation of flows with and without the hypothetical diversion levee lowered the elevations at the downstream end of the study area by only about 0.2 foot. Because the maximum error of 0.2 foot at the downstream boundary near Lehmberg Road was minimal, the downstream boundary conditions were not changed for each flow condition for the flood discharge being simulated. A profile-convergence study showed that profiles converged by 1,000ee upstream of the downstream boundary with a 0.2-foot starting error. Therefore, the slight errors in downstream starting water-surface elevations introduced by the varying discharges approaching Lehmberg Road for an individual flood do not affect the solutions near the U.S. Highway 82 bridge.

The boundary conditions representing the Magby Creek distributary are water-surface elevations and were based on average flow depths. The flow depths in the flood plain near the U.S. Highway 82 bridge were fairly uniform and the flow depths were assumed to be similar at the boundary representing the Magby Creek distributary. Computed elevations under the highway bridge using the 1990 flood discharges agreed within 0.1 foot of the recorded high-water elevations under the bridge. This close agreement between simulated and recorded elevations at the highway bridge indicates that the assumption of uniform flow depths at the open boundary representing the Magby Creek distributary was appropriate.

Calibration Strategy

The calibration strategies involved in this study are based on interpretive applications of basic hydraulic principles. Hydraulic data needed for proper calibration of the FESWMS model are limited because no floods occurred during the study. However, a few high-water marks were recovered after the floods of February and December 1990. The discharges for both of these floods were estimated [using the areal study technique as described in the USGS report by Rantz and others (1982)] to be about 2,800 cubic feet per second. It is possible that peak flood discharges estimated using the areal study technique have errors within plus or minus 25 percent of more.

Because of insufficient data, parameters for model calibration were estimated on the basis of engineering judgment. After the calibrated model converged upon a solution, no parameters were adjusted. The parameters representing eddy viscosities were set to 100 square feet per second for the 1990 flood discharges and 400 square feet per second for the 50- and 100-year discharges. Previous studies indicated that because of the lack of information for the exact values of the eddy viscosity parameters, the relatively high values stated above are adequate and any adjustment to the viscosity parameter has minimal effects upon the final solution after the model converges (Gilbert and Froehlich 1987). With limited high-water marks and an estimated discharge, the model could not be calibrated as well as desired. However, the simulated elevations did adequately correspond with the surveyed high-water elevations which indicates that the finite-element grid, the selected roughness coefficients, the selected model parameters, and the estimated discharge were reasonable.

The finite-element grid and calibration parameter values used in the simulations of the 1990 floods were also used for simulations of the 50- and 100-year floods. The 50- and 100-year flood discharges are used for the design of hydraulic structures, but there are no independent calibration data for these floods. The results of the simulations for the 50- and 100-year floods should be judged accordingly.

A convergence study was performed in which the original finite-element grid used for flow simulations (Figure 2) was refined by a factor of about 4. Each quadrangular element was subdivided into four similar elements and each triangular element was divided into three similar elements and the 1990 floods with all ponds in place were simulated. The 1990 flood simulations used 3,872 elements and 18,030 nodes with the refined grid. On average, the simulated discharges flowing around the pond levees using the refined grid agreed within 3.5 percent with the results obtained when using the original grid. When using the refined grid, the computed water-surface elevations agreed within 0.1 foot of the water-surface elevations computed using the original grid. The maximum difference in water-surface elevations when comparing the results obtained from using the two grids is about 0.3 foot and occurs in two areas upstream of Old Mill Road. Because the results obtained when using the refined grid agreed closely with those obtained when using the original grid, it was concluded that the original grid was adequate for flow simulations in this study.

Model Validation

The proper technique of validating a calibrated model is to simulate an independent hydraulic event for which the discharge and elevations are known without adjusting model parameters determined for the original calibration event. If the solution is comparable to the measured data for the independent event, the model probably is valid.

Because the two floods of 1990 were nearly identical in discharges and elevations, it was not possible to validate the calibrated model. However, it was assumed that the model was calibrated as well as possible because: 1) of the close agreement between surveyed and simulated elevations as stated above, and 2) simulations of the 1990 floodflows correctly indicated the locations of damage to the pond levees where overtopping occurred.

SIMULATION OF FLOODFLOWS

Six floodflow scenarios were simulated and categorized according to discharges as the 1990 floods, the 50-year flood, or the 100-year flood. In the

50- and 100-year discharge categories, floodflows were simulated with and without a hypothetical floodflow diversion levee connecting the east levee of pond 2 to the west spur dike of the U.S. Highway 82 bridge. The 1990 floodflows were also simulated and the pond 3 levees were removed in one simulation to represent present conditions.

Floods of February and December 1990

From the areal study technique, the peak discharges for the floods of 1990 were estimated to be about 2,800 cubic feet per second. The recurrence interval for this discharge is about 3 years, which means a similar flood has a 33 percent chance of being equaled or exceeded in any given year.

During the floods of 1990, the U.S. Highway 82 embankments were in place. The six ponds located on private land near the north embankment of U.S. Highway 82 were also in existence during this time and some pond levees were slightly damaged by the flows.

The 1990 floods were not severe enough to inundate the entire Magby Creek flood plain. It appears that the flows were maintained near the main channel of Magby Creek until they were separated at the Magby Creek distributary. Eyewitnesses state that at Old Mill Road, the Magby Creek channel was flowing bank-full and the Magby Creek distributary was out of its banks. Conveyance computations indicate that at the point of flow separation at the distributary when Magby Creek is flowing full, the Magby Creek channel is capable of conveying 1,400 cubic feet per second. Therefore, it is concluded that the flows separated at the distributary with about 1,400 cubic feet per second flowing in the Magby Creek channel and about 1,400 cubic feet per second flowing in the Magby Creek distributary flood plain.

Because the Magby Creek flood plain was not inundated during the 1990 floods, there was no intermingling flow between the Magby Creek and the Magby Creek distributary flood plains. Therefore, the elements representing the Magby Creek channel and flood plain were turned off and only flows in the Magby Creek distributary flood plain were simulated (1,400 cubic feet per second). The following simulations represent the flow conditions for the floods of 1990 within the Magby Creek distributary flood plain with alterations depicting observed, present, and possible scenarios. In the following sections, the percentage of the total flood discharge of 2,800 cubic feet per second is stated in parenthesis following the computed discharge.

Flows for 1990 Conditions

Floodflows were simulated depicting the floods of February and December 1990. Flow simulations of the 1990 floods indicate that a maximum discharge of 400 cubic feet per second (14 percent) flowed south of pond 4, pond 5, and pond 6 at the U.S. Highway 82 embankment in the riprap channel constructed by the MDOT near the upstream end of the study area. Simulations indicate that about 600 cubic feet per second (21 percent) flowed through the Magby Creek distributary bridge at U.S. Highway 82. The remaining 800 cubic feet per second (29 percent) flowed in the southern one-half of the Magby Creek flood plain toward Lehmberg Road (Figure 3).

High-water marks were surveyed at the Magby Creek distributary bridge at Old Mill Road and were 201.5 and 201.1 feet at the upstream and downstream sides, respectively. The average elevation of high-water marks under the U.S. Highway 82 bridge was 197.6 feet. The elevations determined from the simulations of the 1990 floods were 202.0 and 201.6 feet at the upstream and downstream sides of the Old Mill Road bridge, respectively. The simulated elevation at the downstream side of the U.S. Highway 82 bridge was 197.7 feet.

Velocity vectors near the U.S. Highway 82 bridge were also plotted so computed flow directions for the 1990 floods could be visually inspected (Figure 4). Simulations indicate that the flows separated and flowed around the pond 3 levees. During the 1990 floods, simulations indicate that the flows north of pond 3 (about 1,050 cubic feet per second) separated with about 540 cubic feet per second flowing north of pond 2 and about 510 cubic feet per second flowing between pond 2 and pond 3. Simulations indicate the flows passing between the west spur dike of the highway bridge and the southeastern corner of pond 2 were about 260 cubic feet per second (9 percent).

Flows with Pond 3 Removed

The owner of pond 3 has removed the levees from around that pond since the floods of 1990; therefore, the 1990 floods were simulated with the pond 3 levees removed. If no changes are made within the flood plain or the U.S. Highway 82 bridge opening, the flows simulated without pond 3 would occur during a flood similar to those of 1990.

Removal of the levees surrounding pond 3 had no effect on elevations and flow distributions upstream of Old Mill Road when simulating the floods of 1990. Therefore, the control for the 1990 floodflows in the study area upstream of Old Mill Road was the Magby Creek distributary bridge at Old Mill Road.

With flows not being as confined near the highway embankment upstream of the U.S. Highway 82 bridge by the pond 3 levees, the simulated velocities were not as large in the flows that were prone to bypass the bridge. The simulations indicate about an 8 percent increase in discharge (50 cubic feet per second) would be conveyed through the bridge without the pond 3 levees in place for a total of 650 cubic feet per second (23 percent). About 300 cubic feet per second (11 percent) would flow between the west spur dike and the southeastern corner of pond 2 and about 450 cubic feet per second (16 percent) would flow around the northern side of pond 2. A total discharge of about 750 cubic feet per second would continue in the southern one-half of the Magby Creek flood plain to approach Lehmberg Road; about 6 percent less than with the pond 3 levees in place.

Flows with the Hypothetical Floodflow Diversion Levee

The 1990 flood discharges with a hypothetical floodflow diversion levee connecting the west spur dike on the U.S. Highway 82 bridge to the east levee of pond 2 were not simulated. Sufficient data had been provided by previous simulations to estimate flow distributions.

Output from previous simulations (flows with pond 3 removed) indicates that during a flood similar to the 1990 floods and assuming that the pond 3 levees are not rebuilt, about 300 cubic feet per second would flow between the west spur dike of the highway bridge and the southeastern corner of pond 2 and about 650 cubic feet per second would flow through the highway bridge. If a levee were constructed connecting the west spur dike to the east levee of pond 2, the 300 cubic feet per second would then flow through the bridge for a total of 950 cubic feet per second (34 percent). Therefore, about 450 cubic feet per second (16 percent) would flow around the north levee of pond 2 toward Lehmberg Road in the southern part of the Magby Creek flood plain.

50-Year Flood

Floodflows were simulated depicting the Magby Creek 50-year flood. The estimated 50-year flood discharge is 8,030 cubic feet per second (Figure 2) and has a 2 percent chance of being equaled or exceeded in any given year. The 50-year flood was selected for flow analyses because it is the primary flood that the MDOT uses in the design of hydraulic structures.

During the 50-year flood, floodwaters would submerge the entire width of the Magby Creek flood plain to an average depth of about 2.4 feet. Because the pond levees are about 1 to 2 feet high, it is assumed that they would be destroyed during a 50-year flood. Therefore, the levees of pond 1 to pond 6 were removed for these simulations.

Flows without the Hypothetical Floodflow Diversion Levee

The 50-year floodflows were simulated with the U.S. Highway 82 embankments in place. Simulations indicate that the 50-year floodflows would overtop Old Mill Road by about 1.0 foot. The average computed water-surface elevations at the upstream and downstream sides of the overtopped section of Old Mill Road are 204.3 and 202.3 feet, respectively; thus, a differential of about 2.0 feet is expected. The weir flow condition at Old Mill Road during the 50-year flood is non-submergence for a 2.0-foot differential.

The discharge across Old Mill Road during a 50-year flood was simulated with 17 weir sections representing roadway embankment elevations and widths. The total length of the weir (perpendicular to the floodflows) is about 1,770 feet. Simulations indicate that the discharge across Old Mill Road during the 50-year flood would be about 5,350 cubic feet per second (67 percent). Because of submergence, flows through the bridges at Old Mill Road were simulated as pressure flows. The simulated discharges at Old Mill Road through the Magby Creek bridge and the Magby Creek distributary bridge were 1,850 cubic feet per second (23 percent) and 830 cubic feet per second (10 percent), respectively.

The simulations indicate that about 360 cubic feet per second (4 percent) of the 8,030 cubic feet per second would pass through the U.S. Highway 82 bridge. The remaining 7,670 cubic feet per second (96 percent) would flow downstream toward Lehmberg Road.

Flows with the Hypothetical Floodflow Diversion Levee

The 50-year floodflows were simulated with a hypothetical floodflow diversion levee connecting the west spur dike on the U.S. Highway 82 bridge to the east levee of pond 2. For flow simulations, it was assumed that the diversion levee and the east levee of pond 2 were high enough to avoid overtopping by floodflows. The east levee of pond 2 extends about 1,000 feet into the Magby Creek flood plain from the west spur dike.

Simulations indicate that flow distributions and water-surface elevations upstream of Old Mill Road would remain unchanged by the construction of the floodflow diversion levee. However, the diversion levee would divert more floodflows from the Magby Creek flood plain through the U.S. Highway 82 bridge. With the diversion levee in place, about 1,500 cubic feet per second (19 percent) would flow through the U.S. Highway 82 bridge and about 6,530 cubic feet per second (81 percent) would flow down the Magby Creek flood plain toward Lehmberg Road.

100-Year Flood

Floodflows were also simulated depicting the Magby Creek 100-year flood. The estimated 100-year flood discharge is 9,130 cubic feet per second and has a 1 percent chance of being equaled or exceeded in any given year. The Federal Highway Administration requires hydraulic structures to allow a 100-year flood to pass and to meet specified backwater limitations.

During the 100-year flood, floodwaters would submerge the entire width of the Magby Creek flood plain to an average depth of about 2.7 feet. Because the pond levees are about 1 to 2 feet high, it is assumed that they would be destroyed during a 100-year flood. Therefore, the levees of pond 1 to pond 6 were removed for these simulations.

Flows without the Hypothetical Floodflow Diversion Levee

The 100-year floodflows were simulated with the U.S. Highway 82 embankments in place. Simulation results indicate that the 100-year floodflows would overtop Old Mill Road by about 1.1 feet. The average computed water-surface elevations at the upstream and downstream sides of the overtopped section of Old Mill Road are 204.4 and 202.5 feet, respectively; thus, a differential of about 1.9 feet is expected. The weir flow condition at Old Mill Road during the 100-year flood is non-submergence for a 1.9-foot differential.

The discharge across Old Mill Road during a 100-year flood was simulated with 17 weir sections representing roadway embankment elevations and widths. The total length of the weir (perpendicular to the floodflows) is about 1,770 feet. Simulations indicate that the discharge across Old Mill Road during the 100-year flood would be about 6,270 cubic feet per second (69 percent). Because of submergence, flows through the bridges at Old Mill Road were simulated as pressure flows. The simulated discharges at Old Mill Road through the Magby Creek bridge and the Magby Creek distributary bridge were 2,000 cubic feet per second (22 percent) and 860 cubic feet per second (9 percent), respectively.

The simulations indicate that about 410 cubic feet per second (4 percent) of the 9,130 cubic feet per second would pass through the U.S. Highway 82 bridge. The remaining 8,720 cubic feet per second would flow down the Magby Creek flood plain toward Lehmberg Road.

Flows with the Hypothetical Floodflow Diversion Levee

The 100-year floodflows were simulated with a hypothetical floodflow diversion levee in place connecting the west spur dike on the U.S. Highway 82 bridge to the east levee of pond 2. For flow simulations, it was assumed that the connecting diversion levee and the east levee of pond 2 were high enough to avoid overtopping by floodflows.

Simulations indicate that the flows upstream of Old Mill Road would remain unchanged by the construction of the floodflow diversion levee. However, the diversion levee would divert more floodflows from the Magby Creek flood plain through the U.S. Highway 82 bridge. With the diversion levee in place, about 1,650 cubic feet per second (18 percent) of the 9,130 cubic feet per second would flow through the U.S. Highway 82 Magby Creek distributary bridge and about 7,480 cubic feet per second (82 percent) would flow down the Magby Creek flood plain toward Lehmberg Road.

Simulations of the 50- and 100-year floods with a hypothetical floodflow diversion levee connecting the west spur dike of the highway bridge to the east levee of pond 2 indicate that discharges in the Magby Creek distributary flood plain would increase by about 300 percent above present conditions. Flows in the Magby Creek flood plain presently approaching Lehmberg Road would decrease by about 14 percent.

SUMMARY AND CONCLUSIONS

A two-dimensional finite-element surface-water model was used to study the potential effects of a hypothetical floodflow diversion levee on floodflows in the Magby Creek basin east of Columbus, Mississippi. Scenarios with and without the hypothetical floodflow diversion levee in place were simulated for the 50- and 100-year design floods. Floodflows which occurred during the 1990 floods were also simulated. In addition, floodflows without the pond 3 levees were simulated to represent present conditions.

Simulations of floodflows indicate that the hypothetical diversion levee would increase flows in the Magby Creek distributary flood plain by about 46 percent during a flood similar to the 1990 floods. The corresponding decrease in flows approaching Lehmberg Road is about 19 percent. Simulations indicate that the hypothetical floodflow diversion levee would increase flows in the Magby Creek distributary flood plain by about 300 percent during the 50- and 100-year floods. The resulting average decrease in flows approaching Lehmberg Road would be about 14 percent.

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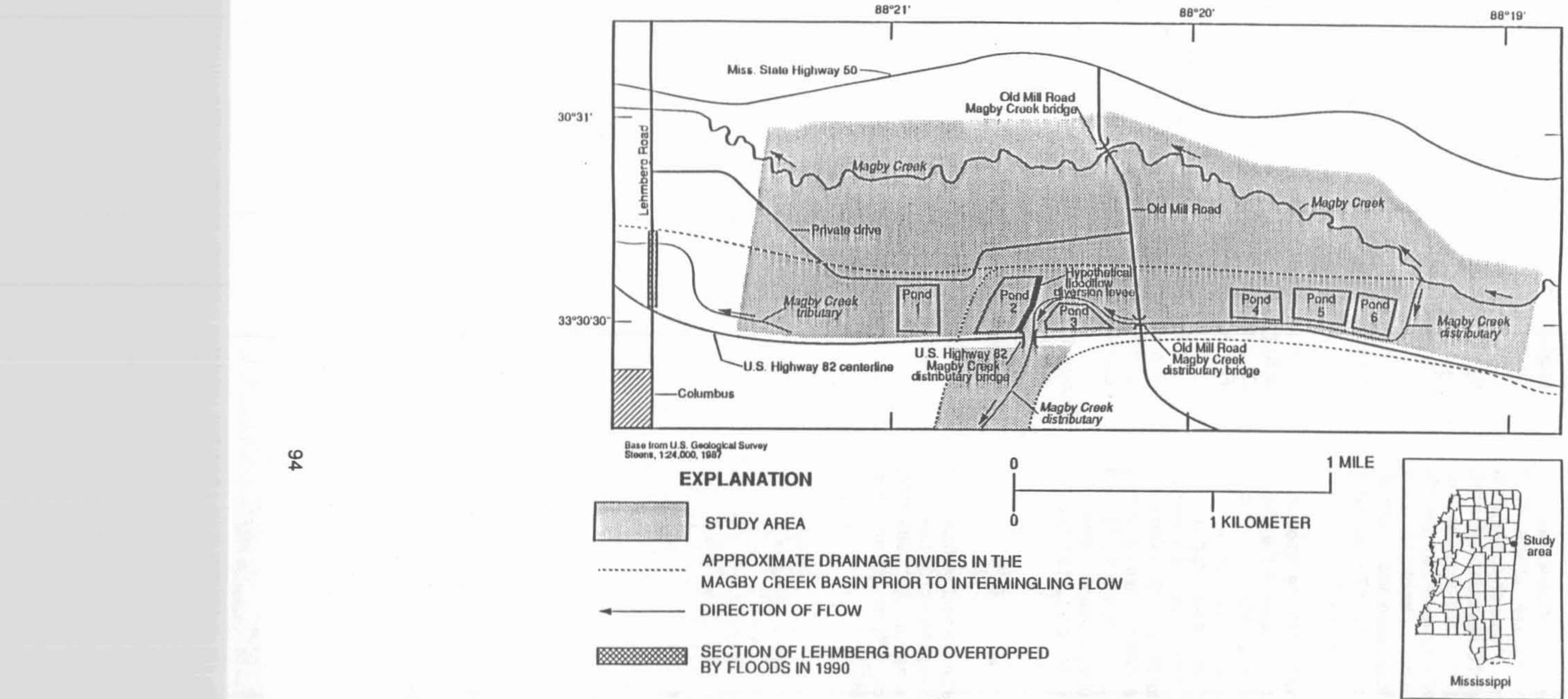
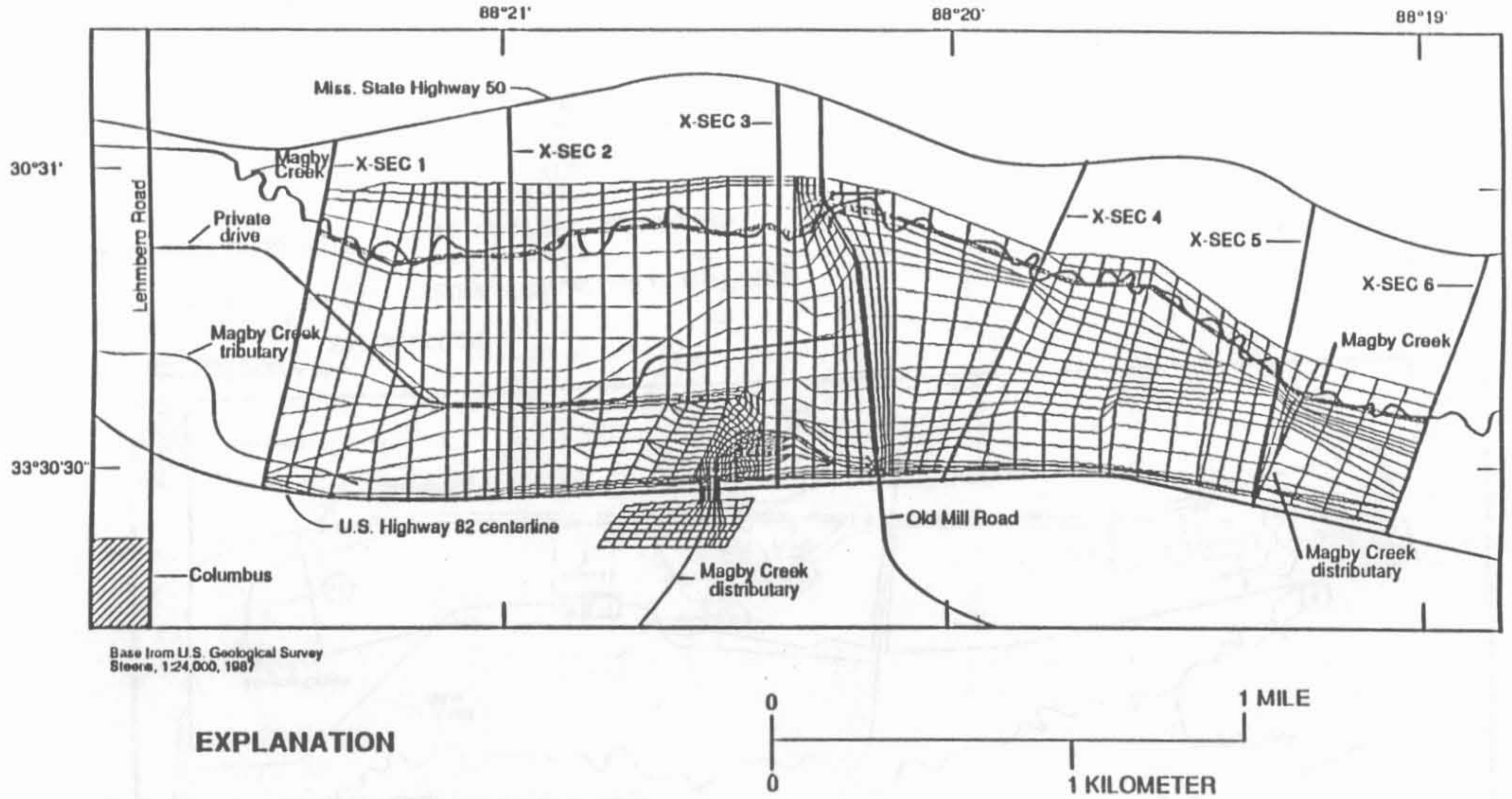


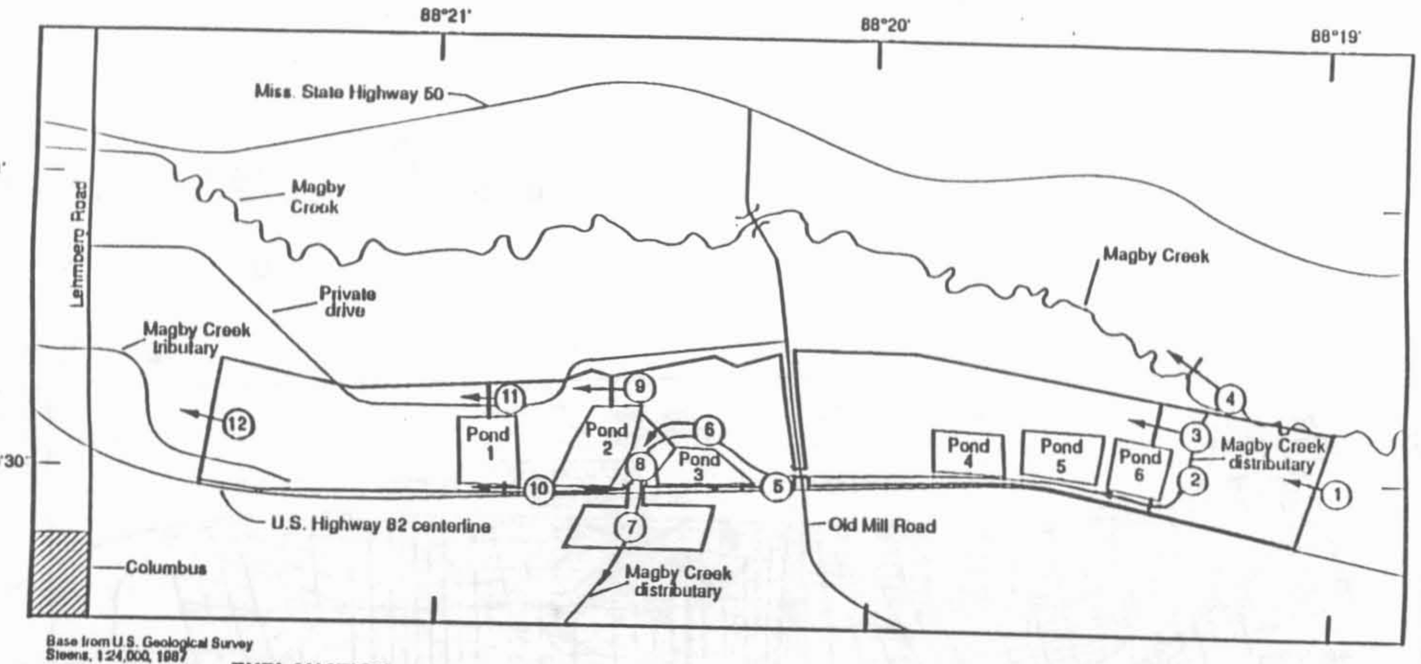
Figure 1. -- Location of study area.



EXPLANATION

X-SEC 1 — MISSISSIPPI STATE DEPARTMENT OF TRANSPORTATION CROSS SECTION NUMBER AND LOCATION

Figure 2. -- Finite element grid used in flow simulations.



EXPLANATION

FLOW SECTION AND DIRECTION

0 — 1 MILE

0 — 1 KILOMETER

FLOW SECTION NUMBER	DISCHARGE, IN CUBIC FEET PER SECOND
1	1,400
2	400
3	1,000
4	1,400
5	350
6	510
7	600
8	260
9	540
10	200
11	600
12	800

Figure 3. -- Computed flow distributions for the floods of February and December 1990.

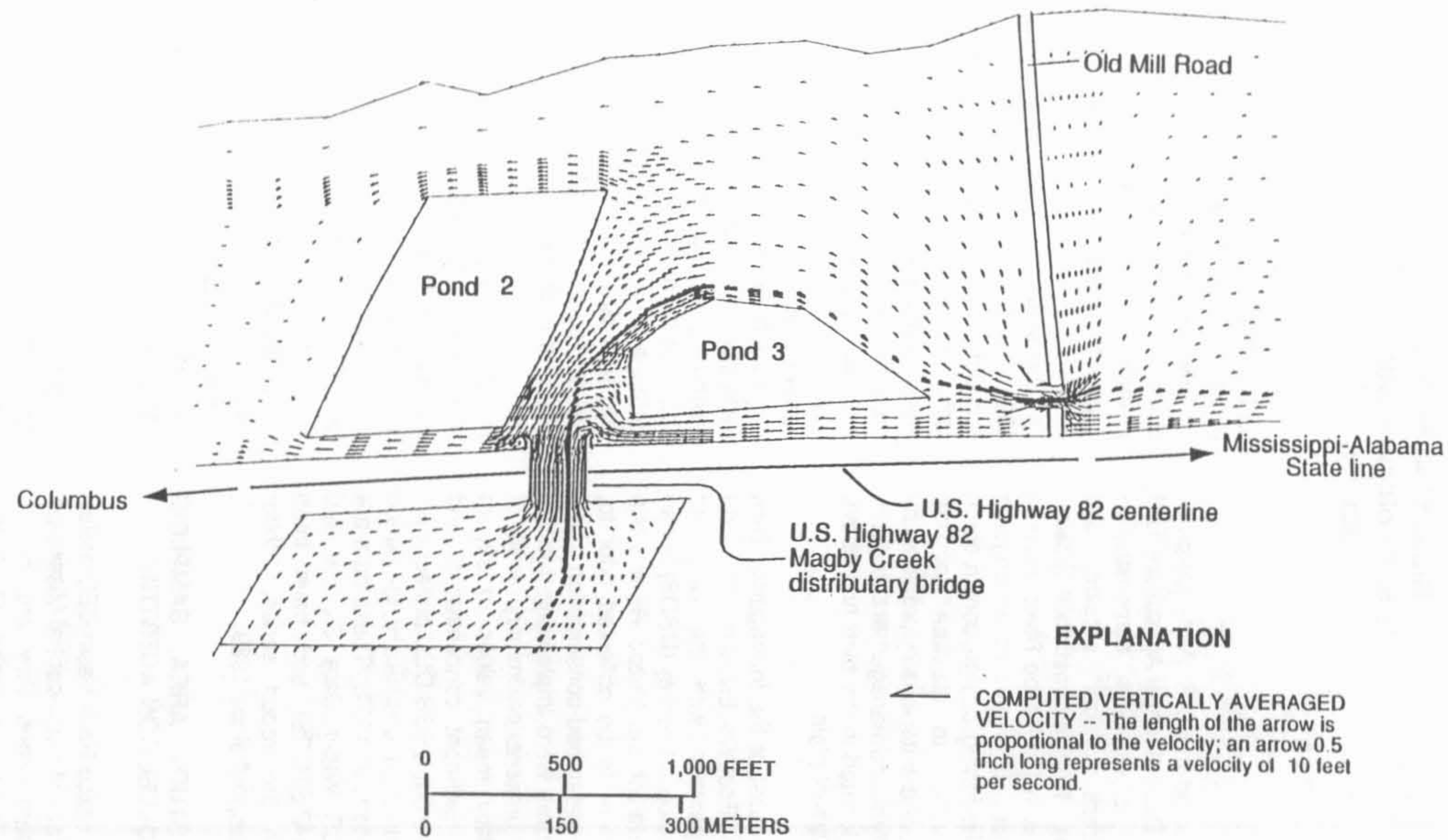


Figure 4. -- Computed velocity vectors near the U.S. Highway 82 bridge for the floods of February and December 1990.